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# DESIGN OF STEEL STRUCTURES TO RESIST THE EFFECTS OF HE EXPLOSIONS

John Healey, et al

Ammann and Whitney

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Picatinny Arsenal

August 1975

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# DESIGN OF STEEL STRUCTURES TO RESIST THE EFFECTS OF HE EXPLOSIONS

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AUGUST 1975

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This report contains criteria and procedures for the design and analysis of steel structures and structural elements subjected to blast pressures produced by HE explosions. The primary emphasis is on structural steel applications for acceptor structures located in the low to intermediate pressure level range. This report on steel design parallels the material on reinforced concrete in the tri-service design manual "Structures to Resist the Effects of Accidental Explosions", TM 5-1300, and is intended to be used in conjunction with that manual.
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Criteria for dynamic plastic design are presented including recommended design stresses, maximum inelastic deformations and maximum total displacements. Design procedures are given for steel beams, plates, columns, and beam-columns, together with special requirements for blast doors, cold-formed steel roof and wall panels, open-web joists, and connections. The preliminary design of single-story, multi-bay frames is also covered. Illustrative design examples are presented.

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# SUMMARY

This report was developed as part of a program undertaken by the Manufacturing Technology Directorate of Picatinny Arsenal in recognition of the need for expanded design information pertaining to structures subjected to blast environments due to accidental explosions within ammunition facilities. The purpose herein is to provide facility designers with criteria and procedures for the design of steel structures and structural elements. The report is intended to be used in conjunction with the tri-service design manual, "Structures to Resist the Effects of Accidental Explosions" (TM 5-1300), and the AISC "Manual of Steel Construction".

The dynamic behavior of steel structures, elements and connections is discussed and criteria for dynamic plastic design are presented, namely, design stresses, acceptable levels of inelastic response, and maximum acceptable deformations. The criteria were developed considering structural integrity, safety for personnel and sensitive equipment during the accident, and the post-accident condition of the structure.

Detailed procedures are given for the design of steel beams, plates, columns and beam-columns, together with special requirements for blast doors, cold-formed steel roof and wall panels, open-web joists and connections. The preliminary design of multi-bay frames is treated in detail. A series of illustrative design examples is provided.

# CONCLUSIONS AND RECOMMENDATIONS

The criteria and procedures presented in this report provide a rational basis for the analysis and design of steel structures and structural elements subjected to a blast environment generated by a high-explosive detonation. The primary emphasis is on structural steel applications for acceptor structures located in the low to intermediate pressure level range.

It is recommended that this material be implemented in the blast-resistant design of structures within facilities for the manufacture and storage of explosive materials.

# CHAPTER 1

# INTRODUCTION

## 1.1 Background

Special procedures and criteria are necessary in the design of facilities for the manufacture, maintenance, modification, inspection and storage of explosive materials in order to avoid mass detonations and explosion propagation in the event of an accidental explosion and to ensure protection for personnel and equipment. The basic design document in this area is the tri-service manual, "Structures to Resist the Effects of Accidental Explosions" (TM 5-1300). This manual contains comprehensive information on the principles of protective design, the calculation of blast loading, dynamic analysis and detailed procedures for designing reinforced concrete protective structures and provides a sound basis for facility design and review.

However, since TM 5-1300 is directed primarily to the design of reinforced concrete, supplementary material is required to treat in detail other structure types and materials of construction employed in the design of modern ammunition facilities. An important area where additional information is needed is the design of steel structures and steel elements. The Manufacturing Technology Directorate of Picatinny Arsenal, as part of its overall Safety Engineering Support Program for the U.S. Army Armament Command, has undertaken the preparation of this report with the assistance of Ammann & Whitney, Consulting Engineers, in order to fill this need.

## 1.2 Objective and Scope

The basic purpose of this report is to provide ammunition facility designers with criteria and procedures for the design of steel protective structures parallel to the specific information on reinforced concrete structures presented in TM 5-1300. In this way, a rational basis will be available for providing blast protection through the use of structural steel for both close-in structures and structures removed from the immediate vicinity of the blast.

This report has been prepared with the following general guidelines:

(a) The report shall be consistent in philosophy and format with TM 5-1300 with modifications as necessary to account for the particular requirements of steel design.

Furthermore, it is assumed that this report will be used in conjunction with TM 5-1300 by designers possessing a basic familiarity with the contents of that manual.

(b) The detailed provisions for inelastic blast-resistant design of steel elements and structures shall be consistent with conventional static plastic design procedures as presented in the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" and modified as required to account for blast loading resulting from high explosive detonations.

(c) In all cases, the static provisions of the AISC Specification represent a minimum requirement for conventional dead and live loads. Moreover, it is presumed that designers using this report are familiar with static plastic design procedures for steel.

(d) With regard to the design and analysis of multi-degree-of-freedom steel frames, it is assumed that this report will be supplemented with an elasto-plastic frame analysis computer program such as the Program DYNFA developed as part of this overall effort for Picatinny Arsenal by Ammann & Whitney.

(e) While material related to close-in effects is included in this report, the primary emphasis is on applications where the structure is located a distance from the blast area.

In cases of low to intermediate blast pressures, the design objective is the protection of personnel and valuable equipment or material, while at the same time providing an economical solution; hence, in many cases, the designs will involve the use of standard structural shapes and building components strengthened to provide the required blast protection. It is felt that the criteria and procedures provided in this report can be effectively utilized to achieve cost-effective designs in these applications.

The procedures and criteria in this report have been developed based upon state-of-the-art information and data for the behavior of steel elements and structures under blast loading. Hence, use of this material should lead to designs consistent with the design objectives. However, in order to account for normal variations in material quality and workmanship and uncertainties in the prediction of loadings, it is recommended that the charge weight be increased by 20 percent for design, as required in TM 5-1300. Modification of this requirement for particular cases should be approved by the cognizant military construction agency.

Generally speaking, the influence of conventional dead and live loads can be neglected in blast design or in the evaluation of the capacity of a blast-resistant structure. However, the effect of such loads upon the available capacity for blast resistance may be significant in the design of structures for relatively low overpressures, e.g., less than 1.0 psi or in the evaluation of the blast resistance of a structure designed for conventional loads.

## 1.3 Format of the Report

This report is divided into chapters devoted to individual topics related to the design of steel protective structures. For the purpose of economical presentation, the directly applicable material from TM 5-1300 and the AISC Specification and Manual is not repeated in this report. As far as possible, applicable equations, design charts and tables, and commentary material are included herein by reference. However, a brief summary of the information from TM 5-1300 which is applicable to this report is presented in the remainder of this chapter. The following sections contain a qualitative description of protective design concepts and describe the calculation of blast loads, applications of steel structures in protective design and the basic elements of the dynamic response calculations following the procedures developed in TM 5-1300.

Chapter 2 describes the static and dynamic properties of structural steels, outlines the basic philosophy and objectives of blast design and presents recommended design criteria for various types of basic steel elements and frames.

In Chapters 3 and 4, detailed provisions are presented for the plastic design of steel beams, columns and beam-columns under dynamic load. Guidelines are given for treating unsymmetrical bending along with special consideration for blast doors, cold-formed steel floor and wall panels, and open-web joists.

Single-story, multi-bay, rigid frames and frames with supplementary bracing are treated in detail in Chapter 5. A preliminary design procedure is provided for this class of frames due to their wide usage in ammunition facilities.

Recommendations for the design of connections are given in Chapter 6. Structural steel connections are considered along with the special requirements for connections involving cold-formed steel floor and wall panels.

Chapter 7 contains detailed example problems covering the design of beams, lateral bracing, cold-formed steel panels, open-web joists, columns and beam-columns, steel plate blast doors, beams subject to unsymmetrical bending and the preliminary design of a single-story, multi-bay rigid frame.

A series of charts used for determining the elasto-plastic resistance, stiffness and deflection for uniformly loaded two-way elements with various support conditions are given in Appendix A. These charts supplement the material in Chapter 5 of TM 5-1300.

The symbols used in the text are defined in Appendix B.

Appendix C presents examples of framing connections, structural details and blast doors used in structures designed for blast.

## 1.4 Protective Design Concepts and Explosive Effects

### 1.4.1 System Components and Basis for Design

From the viewpoint of protective design, ammunition facilities can be treated as being composed of three principal systems as described in Chapter 2 of TM 5-1300. Briefly, these systems are:

(1) The Donor System which involves the source of the potential hazard. The hazardous output consists primarily of blast pressures; but in certain circumstances, it is necessary to consider fragments.

(2) The Receiver System which represents the items which may potentially receive the output of an explosive accident including personnel, equipment and acceptor explosive materials.

(3) The Protective System which involves the use of protective structures such as barriers and shelters or separation distance designed to provide the receiver with the required level of protection consistent with economy and the operating requirements of the facility.

The design of the overall facility and its individual structures essentially requires that a balance be achieved among functional and operational requirements, loading characteristics, dynamic response, structural capacity, personnel safety, equipment and explosive sensitivity and economics. These various factors, elaborated upon in Chapter 3 of TM 5-1300, thereby constitute the constraints on the design and provide the background and the basis for the detailed design criteria.

The interaction between loading characteristics and structural response upon the details of a design can be expressed most efficiently in terms of the three situations that can be encountered as a function of the proximity of the accident to the structure together with the response characteristics of the structure under design, namely:

(a) Close-in to the explosion - The designs are usually impulse sensitive since the load duration, $t_o$, is considerably shorter than the time required for the structure to reach maximum response, $t_m$.

(b) Intermediate cases where $t_m$ and $t_o$ are of the same order of magnitude and the response of the structure to both impulse and pressure must be considered.

(c) Structures removed from the location of the detonation - The designs are characteristically pressure-sensitive due to load durations considerably longer than the structural response time. An exception to this situation can be frame structures which are flexible and therefore can reach their peak response after the load duration, even in the low pressure region.

Familiarity with these load-response characteristics coupled with a knowledge of the structure's ability for absorbing energy and undergoing deformation are important elements in the design of protective structures. Other factors which must be taken into account in the establishment of the overall system design criteria include:

(a) Protection category - The type of protective structure required is determined by this factor. Shelter structures provide protection to personnel, equipment and sensitive explosives from primary and secondary fragments, blast pressures, structure motions and the release of hazardous materials. Barrier structures provide for the containment of the explosion to the donor cell or for the prevention of communication of the detonation to other areas of the facility.

(b) Receiver sensitivity - This factor dictates the maximum response level consistent with the inherent vulnerability of the particular receiver items which must be considered, e.g., the sensitivity of personnel, equipment and explosives to blast pressures, motions and fragments.

(c) Operating and functional requirements - These include the functional and geometric characteristics consistent with the operation of the facility and the desired post-accident condition of the structure, ranging from a structure at incipient failure to a structure suitable for reuse without major repair following the accident.

### 1.4.2 Explosive Effects

Basic to protective design is the capability of calculating efficiently and accurately the anticipated output of an accidental explosion in terms of the damaging effects, principally blast pressures and occasionally primary and secondary fragments, against which the protective system must be designed. Chapter 4 of TM 5-1300 contains all the necessary design data, in convenient form, for defining the blast pressures and primary fragment characteristics pertinent to conditions existing in explosive manufacturing and storage facilities. The emphasis is upon blast output calculations since it normally controls the design of either donor or acceptor structures. However, since primary fragments may in certain cases control, complete procedures for calculating the probable characteristics of the fragments resulting from the break-up of the explosive container are also necessary. The procedures in Chapter 4 of TM 5-1300 are completely applicable to the needs of this report.

## 1.5 Steel Structures in Protective Design

### 1.5.1 Behavior of Steel Structures and Elements

The economy of facility design generally requires that protective structures be designed to perform in the inelastic response range during an accident. In order to insure the structure's integrity throughout such severe conditions, the facility designer must be cognizant of the various possible failure modes and their inter-relationships. The limiting design values are dictated by the attainment of inelastic deflections and rotations without complete collapse. The amount of inelastic deformation is dependent not only upon the ductility characteristics of the material, but also upon the intended use of the structure following an accident. In order for the structure to maintain such large deformations, steps must be taken to prevent premature failure by either brittle fracture or instability (local or overall). Guidelines and criteria for dealing with these effects are presented in the body of this report.

By way of further introduction to steel design for those already familiar with the protective design procedures of TM 5-1300, some qualitative differences between steel and concrete protective structures and the impact of these differences upon the orientation of this report are summarized briefly below:

(1) In close-in high-impulse design situations, a massive reinforced concrete structure, rather than a steel structure, is generally employed in order to limit deflections.

(2) In contrast to reinforced concrete structures subjected to high-intensity loading and permitted to respond with large inelastic deformations, the principal emphasis of this report is upon steel acceptor structures located far from the detonation. Due to the relatively low pressure loading and the protection requirements for such structures, the design objective is often reusability following the accident.

(3) Steel structures and elements are considerably more slender, both in terms of the overall structure and the components of a typical member cross-section. As a result, the effect of overall and local instability upon the ultimate capacity is an important consideration in steel design.

(4) The amount of rebound in concrete structures is considerably reduced by internal damping (cracking) and is essentially eliminated in cases where large deformations or incipient failure are permitted to occur. In structural steel, however, a larger response in rebound, up to 100 percent, can be obtained for a combination of short duration load and a relatively flexible element. As a result, steel structures require that special provisions be made to account for extreme responses of comparable magnitude in both directions.

(5) The treatment of stress interaction is more of a consideration in steel since each element of the cross-section must be considered subject to a state of combined stresses. In reinforced concrete, the provision of separate steel reinforcement for flexure, shear and torsion enables the designer to consider these stresses as being carried by more or less independent systems.

(6) Special care must be taken in steel design to provide for connection integrity up to the point of maximum response. For example, in order to avoid premature brittle fracture in welded connections, the welding characteristics of the particular grade of steel must be considered and the introduction of any stress concentrations or notches at the joint must be avoided.

### 1.5.2 Applications of Steel Structures

While the primary application of this report is for steel acceptor-type structures, it also applies in the design of steel components in donor structures. For example, blast doors and, under certain circumstances, steel containment cells may be designed using the guidelines in this report. For the most part, as stated previously, the emphasis is upon structures removed from the immediate vicinity of the detonation. These include typical frame structures with beams, columns and beam-columns composed of standard structural shapes and built-up sections. Moreover, in many cases, the relatively low blast pressures suggest the use of standard building components such as open-web joists, prefabricated wall panels and roof decking strengthened and detailed as required to carry the full magnitude of the dynamic loads anticipated (so-called "strengthened frangible construction"). Another economical application can be the use of entire pre-engineered buildings strengthened locally to adapt these designs for low-blast pressures (up to 2 psi) with short duration (less than 10 ms). For blast loadings with much longer durations, the maximum practical pressure level for the use of strengthened pre-engineered buildings is about 0.5 psi.

## 1.6 Dynamic Analysis and Design

### 1.6.1 General

The first step in a dynamic design entails the development of a trial design considering facility requirements, available materials and economy with members sized by a simple preliminary procedure. The next step involves the performance of a dynamic analysis to determine the response of the trial design to the blast and the comparison of the maximum response with the deformation limits specified in Chapter 2. The final design is then determined by achieving an economical balance between stiffness and resistance such that the calculated response under the blast loading lies within the limiting values dictated by the operational requirements of the facility.

### 1.6.2 Analysis Procedures

The dynamic response calculation involves either a single-degree-of-freedom analysis using the response charts in Chapter 6 of TM 5-1300 or, in more complex structures, a multi-degree-of-freedom calculation with the dynamic elasto-plastic frame program.

A single-degree-of-freedom analysis may be performed for the design analysis of either a given structural element or of an element for which a preliminary design has been performed according to procedures in Chapters 3 and 4. Since this type of dynamic analysis is described fully with accompanying charts and tables in TM 5-1300, it will not be duplicated herein. In principle, the structure or structural element is characterized by an idealized bilinear elasto-plastic resistance function and the loading is treated as an idealized triangular pulse with zero rise time. Response charts are presented in Chapter 6 of TM 5-1300 for determining the ratio of the maximum response to the elastic response and the time to maximum response for the initial response. The equations presented for the dynamic reactions are also applicable to this report.

Multi-degree-of-freedom non-linear dynamic analyses of braced and unbraced rigid frames can be performed with the computer program DYNFA. For single-story unbraced multi-bay frames, the procedures in Chapter 5 provide a basis for preliminary design. The computer program is described in detail in the report, "Analysis of Frame Structures Subjected to Blast Overpressures", Manufacturing Technology Directorate, Picatinny Arsenal, Technical Report No. TR 4839, 1975, Unclassified.

# CHAPTER 2

# DESIGN CRITERIA FOR STEEL ELEMENTS AND STRUCTURES

## 2.1 Introduction

The mechanical properties of structural steel elements are presented in this chapter along with recommended dynamic design stresses and acceptable maximum displacements, and plastic deformations.

Within the broad range of steels presently available, the structural steels for plastic design covered by the AISC Specification are reviewed with regard to their use in protective structures subjected to blast loads. The effects of rapidly applied dynamic loads on the mechanical properties of steel as a structural material are considered and these effects are related to the response of the component elements of steel structures.

Design concepts for blast-resistant steel structures are discussed in detail in order to provide the designer of modern ammunition facilities with an overall understanding of the different modes of failure of steel structures and to caution him against sudden or premature failures.

## 2.2 Design Stresses for Steel Elements

### 2.2.1 Structural Steel

Structural steel is known to be a strong and ductile building material. The significant engineering properties of steel are strength expressed in terms of yield stress and ultimate tensile strength, ductility expressed in terms of percent elongation at rupture, and rigidity expressed in terms of modulus of elasticity.

Structural steel generally can be considered as exhibiting a linear stress-strain relationship up to the proportional limit, which is either close to, or identical to, the yield point. Beyond the yield point, it can stretch substantially without appreciable increase in stress, the amount of elongation reaching 10 to 15 times that needed to reach yield, a range that is termed "the yield plateau". Beyond that range, strain hardening occurs, i.e., additional elongation is associated with an increase in stress. After reaching a maximum nominal stress called "the tensile strength", a drop in the nominal stress accompanies further elongation and precedes fracture at an elongation (at rupture) amounting to 20 to 30 percent of the specimen's original length. It is this ability of structural steel to undergo sizable permanent (plastic)

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deformations before fracturing, i.e., its ductility, that makes steel a construction material with the required properties for blast-resistant design.

Some high strength structural steels do not exhibit a sharp, well defined yield plateau, but rather show continuous yielding with a curved stress-strain relation. For those steels, it is generally accepted to define a quantity analogous to the yield point, called "the yield stress", as that stress which would produce a permanent strain of 0.2 percent or a total unit elongation of 0.4 to 0.5 percent. Such steels have, in general, a smaller elongation at rupture and should be used with caution when large ductilities are a prerequisite of design.

There has always been a continuous trend towards the production of stronger and stiffer materials so that members can carry more loads without substantial increases in material mass and, therefore, cost. Steel as a material cannot be made stiffer since its modulus of elasticity remains essentially constant for different types of steel and various loadings. On the other hand, stronger steels in a wide range of yield points and tensile strengths have been developed. Structural steels now available may be grouped by strength and grade of steel as follows:

(1) Structural carbon steels

(2) High-strength low alloy steels

(3) Quenched and tempered carbon steels, and

(4) Other steels, including quenched and tempered alloy steels, proprietary steels and constructional alloy steels for special purposes.

Earlier criteria and specifications for the design of steel structures limited the use of plastic design to steels having a specified minimum yield point not higher than 33 to 36 ksi. For decades ASTM A7 steel, with a specified minimum yield of 33 ksi, was the basic structural steel for buildings and bridges. ASTM A36, a low-cost carbon steel of structural quality with a yield point of 36 ksi, introduced in 1960, combines improved weldability and increased strength, and has replaced A7 steel in common practice. The limitation of plastic design to these relatively low yield stresses was due to the fact that most experimental verification of provisions for plastic design contained in current specifications had used steels in that range of strength.

By 1965, the applicability of those provisions, with only minor modifications, to high-strength low-alloy steels furnished up to a specified yield point of 50 ksi, had been established. In addition, with the advent of ASTM Specification A572 in 1966, further investigations were undertaken which indicated the applicability of the provisions for plastic design to all grades covered by that standard. On the basis of these investigations, the list of these steels covered by ASTM standard specifications has been increased accordingly.

Blast-resistant design is commonly associated with plastic design since protective structures are generally designed with the assumption that they will undergo plastic deformations. Consequently, the steels to be used should at least meet the requirements of the AISC Specification in regard to their adequacy for plastic design.

The following is a list of steels that are admissible in plastic design and that conform to ASTM specifications:

Structural Steel with 36,000 psi Minimum Yield
Point, ASTM A36

High-Strength Low-Alloy Structural Steel, A242

High-Strength Low-Alloy Structural Manganese
Vanadium Steel, ASTM A441

Structural Steel with 42,000 psi Minimum Yield
Point, ASTM A529

High-Strength Low-Alloy Columbium-Vanadium Steels
for Structural Quality, ASTM A572

High-Strength Low-Alloy Structural Steel with
50,000 psi Minimum Yield Point to 4 in. Thick,
ASTM A588.

Both A242 and A441 are available in three grades, 42, 46 and 50, with the thicker members available in the lower grades only.

A572 is available in six grades (42, 45, 50, 55, 60 and 65) again depending on thickness. Though all grades are acceptable for plastic design, the designer of blast-resistant structures should use caution when utilizing steels having a yield point above 55 ksi. Due to the sensitivity of high strength steels to dynamic loading, special attention should be given to connection details and welding procedures in order to avoid notching effects and prevent premature failure by brittle fracture.

A588 comes also in three grades which were specially developed to provide heavier sections in high-strength steel. It is particularly useful for plates up to 4 inches thick with a yield of 50 ksi.

### 2.2.2 Mechanical Properties under Dynamic Loading

The effects of rapid loading on the mechanical behavior of the steel material have been observed and measured in uniaxial tensile stress tests. Under rapidly applied loads, the rate of strain increases and this has a marked influence on the mechanical properties of steel.

Considering the mechanical properties under static loading as a basis, the effects of increasing strain rates are illustrated in Figure 2.1 and can be summarized as follows:

(1) The yield point increases substantially to the dynamic yield stress value.

(2) The yield plateau increases in range.

(3) The modulus of elasticity remains insensitive to the rate of loading.

(4) The ultimate tensile strength increases slightly, but the percentage increase is less than for the yield stress.

(5) The elongation at rupture either remains unchanged or is slightly reduced.

In actual members subjected to blast loading, the dynamic effects resulting from the rapid strain rates may be expressed as a function of the time to reach yielding. In this case, the mechanical behavior depends on both the loading regime and the response of the system which determines the dynamic effect felt by the particular material.

For members made of A7 steel, studies have been made to determine the percentage increase in the yield stress as a function of strain rate or, in other words, the time to reach yielding. For the purposes of this report, the ratios of dynamic to static yield stress for A7 steel are considered to apply equally well to A36 steel.

For primary structural elements and simple frames designed to withstand low to intermediate pressure levels, the time to reach yield ranges between .01 and 0.1 second. The dynamic

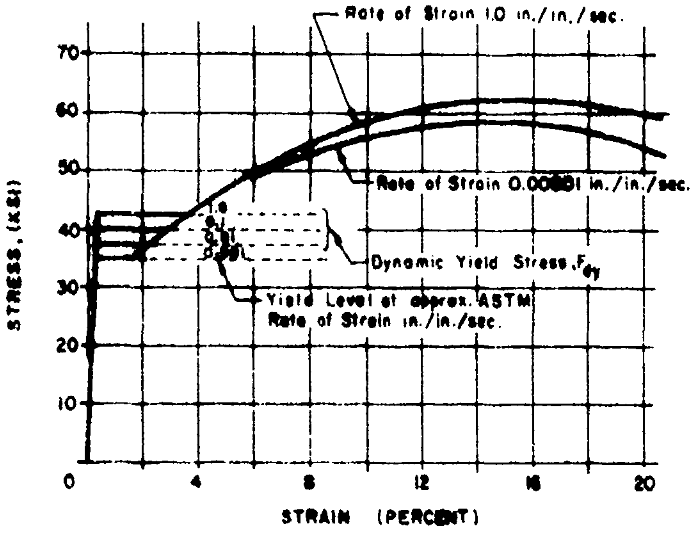


Figure 2.1 Effect of rate of strain on the stress - strain curve of A-7 steel.

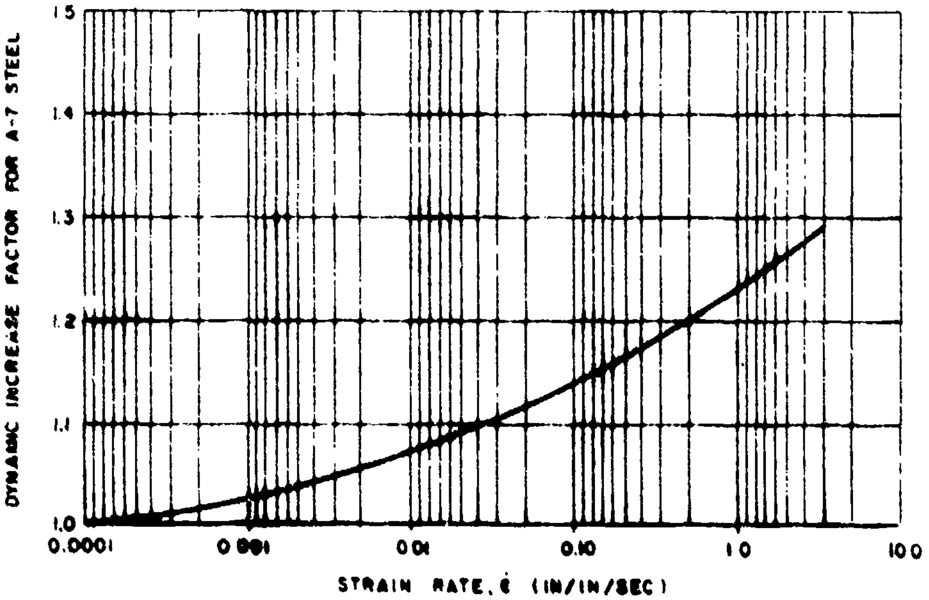


Figure 2.2 Values of dynamic increase factor at various strain rates.

increase factor for these loading rates is about 1.1. However, steel elements, particularly elements designed to withstand high pressure levels, may have much smaller periods. The approximate time to yield under dynamic loading for typical applications is in the range of approximately 0.2 - 0.3 times the fundamental period of vibration of the structure or member being loaded. Using this relation, it follows that for structures with a period of 0.1 second or greater, the ratio of dynamic to static yield stress is 1.2; while for structures with a period of less than 0.1 second, a ratio greater than 1.2 is indicated. The data are presented in graphical form in Figure 2.2 which shows a dynamic increase factor of 1.3 for very rapid loading. From this diagram, a value for the dynamic stress increase factor can be determined based upon the rate of strain corresponding to the response of the structural element. The strain rate, assumed to be a constant value from zero strain to yielding, may be determined according to the following relations:

$$\epsilon = \Delta\epsilon/\Delta t = (\Delta\sigma/E)/\Delta t \qquad (2.1)$$

For high-strength low-alloy steels, a limited number of strain-rate tests show a flattening out of the dynamic yield increase at higher strain rates. For this reason, until more complete data become available, increases above 10 percent are not recommended for high-strength steels.

On the basis of the above, the dynamic increase factors recommended for use in dynamic design are summarized in Table 2.1.

TABLE 2.1 DYNAMIC STRESS INCREASE FACTORS

| Pressure Range | A36 Steel | High-Strength Low-Alloy Steels |
|---|---|---|
| Low to Intermediate | 1.1 | 1.1 |
| High | 1.1 or a higher value as determined from the actual strain rate (Figure 2.2) | 1.1 |

### 2.2.3 Recommended Design Stresses

The yield point of steel under uniaxial tensile stress is generally used as a base to determine yield stresses under other loading states. For instance, the compressive yield stress of steel is equal to $F_y$, the yield point in tension. The shear yield stress is taken as equal to $0.55F_y$.

To determine the plastic strength of a section under dynamic loading, the appropriate dynamic yield stress $F_{dy}$ must be used. In general terms, $F_{dy} = cF_y$, where c is the dynamic increase factor (Table 2.1) and $F_y$ is the yield stress of the steel.

The ASTM value of the yield point for a particular material is a specified minimum value. It has been the practice in design for nuclear blast to use an average yield point which is generally higher than the prescribed minimum. However, in the design of steel structures for ammunition facilities, the probability of occurrence of the load at least once is quite high. In addition, in many cases for operational requirements, the structure is to be reusable. These factors call for a more conservative approach; therefore, it is recommended that the specified minimum value for the yield stress be used as a basis for computation.

It should be noted that if the actual yield stress is greater than that assumed in design, the member will, in fact, exhibit a larger resistance leading to an increase in dynamic reactions. Since the average yield stress is about 15 percent higher than the minimum for A36 steels and about 5 percent for high strength steels, one can anticipate a 10 percent possible increase in actual resistance. However, because shear forces are computed as a function of maximum resistance rather than by means of dynamic reaction, an adequate margin actually exists and could compensate for any possible increase.

To summarize, the dynamic yield stress $F_{dy}$ is to be equal to the dynamic increase factor times the specified minimum yield stress of the steel. The dynamic yielding stress in shear, $F_{dv}$, is taken equal to $0.55F_{dy}$.

$$F_{dy} = cF_y \qquad (2.2)$$

$$F_{dv} = 0.55F_{dy} \qquad (2.3)$$

The dynamic yield stresses for rivets, bolts and welds shall be taken equal to 1.87 times the allowable stresses given in Part 1 of the AISC Specification. This factor accounts for the safety factor of 1.7 used in deriving these allowable stresses and includes a dynamic increase factor of 1.1. By the same reasoning, the tabulated working loads in Part 4 of the AISC Manual may be increased by a factor of 1.87 for use in dynamic design.

## 2.3 Criteria for Ductile Mode Response

### 2.3.1 Introduction

Although plastic behavior is not generally permissible under service loading conditions, it is quite appropriate for design when the structure is subjected to a severe dynamic loading only once or at most a few times during its existence. Under blast pressures, it will usually be uneconomical to design a structure to remain elastic and, as a result, plastic behavior is normally anticipated in order to utilize more fully the energy-absorbing capacity of blast-resistant structures. Plastic design for flexure is based on the assumption that the structure or member resistance is fully developed with the formation of totally plastified sections at the most highly stressed locations. To be consistent with the design assumptions, the actual structure should be proportioned in such a manner as to assure its ductile behavior up to the limit of its load-carrying capacity. This mode of behavior, based on flexural performance, wherein the structure is permitted to develop its full plastic capacity without premature impairment of strength due to secondary effects, such as brittle fracture or instability, is termed "the primary or ductile mode of response".

Another aspect of dynamic design of steel structures subjected to blast loadings is the question of rebound. Unlike the conditions prevailing in reinforced concrete structures where rebound considerations are not of primary concern, steel structures will be subjected to relatively large stress reversals caused by rebound and will require lateral bracing of unstayed compression flanges which were formerly in tension. Rebound is more critical for members supporting light dead loads and subjected to blast pressures of short duration.

In this context, structural resistance is determined on the basis of plastic design concepts, taking into account dynamic yield strength values. The design proceeds with the basic objective that the computed deformations of either the individual members or the structure as a whole, due to the anticipated blast loading, should be limited to prescribed maximum values consistent with safety and the desired post-accident condition.

Deformation criteria are specified in some detail for two different levels of damage for acceptor-type structures located in the low to intermediate pressure level ranges. The maximum deformations specified for both categories are consistent with maintaining structural integrity into the plastic range and with providing safety for personnel and equipment. The distinction between the two design categories pertains to the amount of deformation sustained during the blast loading and consequently, the post-accident condition of the structure.

Structures in the first category, "reusable" structures, are intended to sustain light damage such that they are reusable with only minor repair. Permanent deformations can be tolerated to the extent that they are compatible with future structural safety and with the intended function of the building, including any manufacturing operations which interface with the structure.

The second category includes structures designed to provide safety and structural integrity during the accident but permitted to sustain moderate to severe damage. In this case, however, the damage is such that the post-accident condition is not compatible with future structural safety and the damage is such that the repair work necessary to restore the structure would be excessive. Such structures are, therefore, "non-reusable".

It should be noted that a reusable structure will generally be non-reusable after being subjected to a second accident. Consequently, if it is intended for a structure to be reusable after more than two accidents, the maximum deformation limits specified in Section 2.3.3 should be reduced.

In any event, it is recognized that the post-accident condition of any structure, however designed, will be thoroughly evaluated on an individual basis prior to a decision regarding its suitability for repair and reuse.

These design categories are not intended to cover the severe conditions associated with a donor structure or structures located close-in to a blast. In such cases where the design objective is the prevention of explosion propagation or the prevention of missile generation, the structure may be allowed to approach incipient failure, and deformations well into the strain-hardening range may be permitted for energy absorption.

### 2.3.2 Deformation Criteria - General

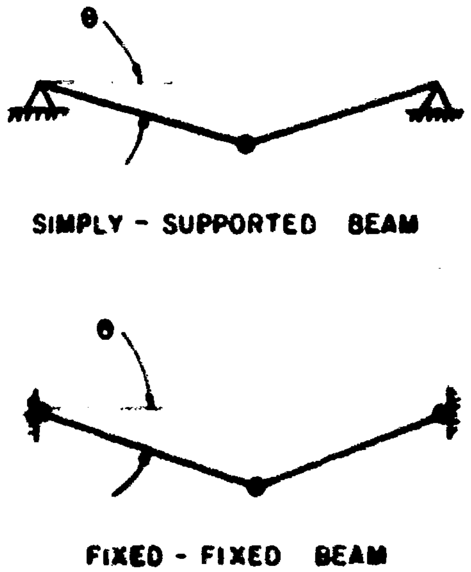
In order to restrict the amount of damage to a structure or element during the process of resisting the effects of an accidental explosion, limiting values must be assigned to appropriate response quantities. Generally speaking, two different types of values will be specified, namely: limits on the level of inelastic dynamic response and limits on the maximum deflections and rotations.

For elements which can be represented as single-degree-of-freedom systems such as beams, floor and wall panels, open-web joists, and plates, the appropriate quantities are taken as the maximum ductility ratio and the maximum rotation at an end support. Following the development in Section IV, Chapter 5 of TM 5-1300, the ductility ratio, μ, is defined as the ratio of the maximum deflection ($X_m$) to the equivalent elastic deflection ($X_E$) corresponding to the development of the limiting resistance on the bilinear resistance diagram for the element. Thus a μ of 3 corresponds to a maximum dynamic response three times the equivalent elastic response. The maximum rotation at an end support, θ, is illustrated in Figure 2.3(a). As shown, θ is the angle between the chord joining the member ends and the chord joining the support and a point on the element where the deflection is a maximum.

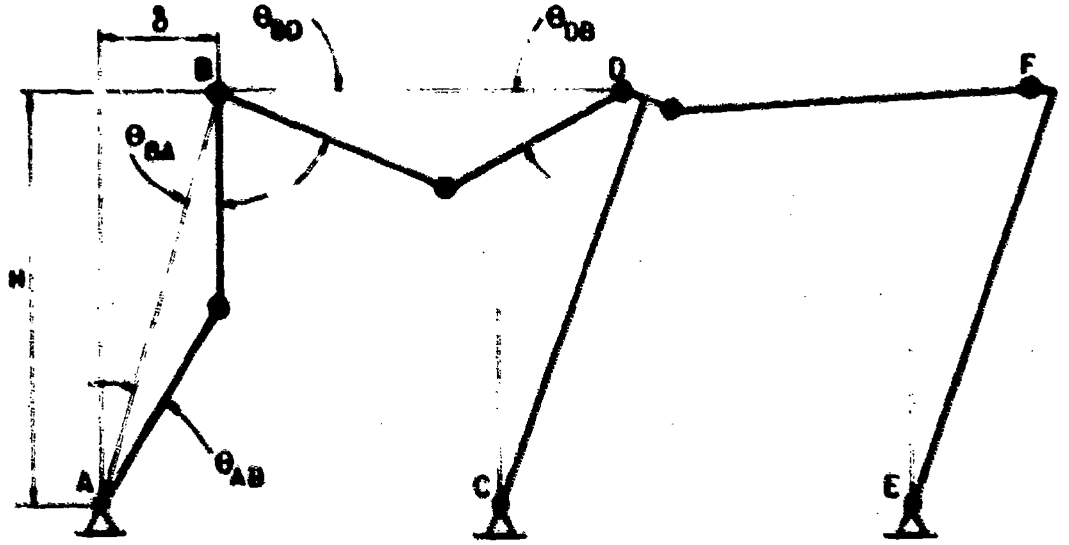
In the detailed analysis of a frame structure, representation of the response by a single quantity is not possible. This fact combined with the wide range and time-varying nature of the end conditions of the individual frame members makes the concept of ductility ratio intractable. Hence, for this case, the response quantities referred to in the criteria are the sidesway deflection of each story and the end rotation, θ, of the individual members with reference to a chord joining the member ends, as illustrated in Figure 2.3(b). In addition, in lieu of a ductility ratio criterion, the amount of inelastic deformation is restricted by means of a limitation on the total rotation permitted for those members with the smaller span-to-depth ratios. For members which are not loaded between their ends, such as an interior column, θ is zero and only the sidesway criteria must be considered. These response quantities, sidesway deflection and end rotation, are part of the required output of the computer program DYNFA which was developed to perform the inelastic multi-degree-of-freedom analysis of frame structures. The designer can use this output to check the sidesway deflection of each story and the maximum rotation at the end of each member.

Section 2.3.3 presents a summary of the design criteria for the reusable and non-reusable design categories for beam elements, frames, plates, cold-formed floor and wall panels and open-web joists. In the remainder of this section, the criteria for beams will be discussed in further detail.

In the case of individual beam elements where ductility ratios as high as 20 are observed at collapse, a maximum ductility ratio of 3 for a reusable structure is specified in order to limit damage to a level for which repair is economically feasible. For a structure which is to be designed as non-reusable after an accident, the structure can be permitted to deflect to a ductility


θ
SIMPLY - SUPPORTED BEAM
θ
FIXED - FIXED BEAM

(a) SIMPLE ELEMENTS


δ
$\theta_{BD}$
$\theta_{DB}$
B
D
F
$\theta_{BA}$
H
A
$\theta_{AB}$
C
E

(b) FRAME STRUCTURE

Figure 2.3 Member end rotation for simple elements and frames.

ratio of 6. In addition, limiting support rotations of 1° and 2° are specified for the reusable and non-reusable cases, respectively. These limiting rotations are assigned as reasonable estimates of the absolute magnitude of end support rotations consistent with the objectives of the two design criteria levels as described in Section 2.3.1.

Figures 2.4(a) and 2.4(b) illustrate the interrelationship between these limiting values of ductility ratio and support rotation for the case of doubly symmetric I- and W-shaped sections with $F_y = 36$ ksi and $F_y = 50$ ksi. These results show the variation of the deflection-span ratio versus the span-depth ratio for simply-supported and fixed-ended beams. For each type of support condition, curves for three ductility ratios are shown, namely:

$\mu = 1$ Elastic design

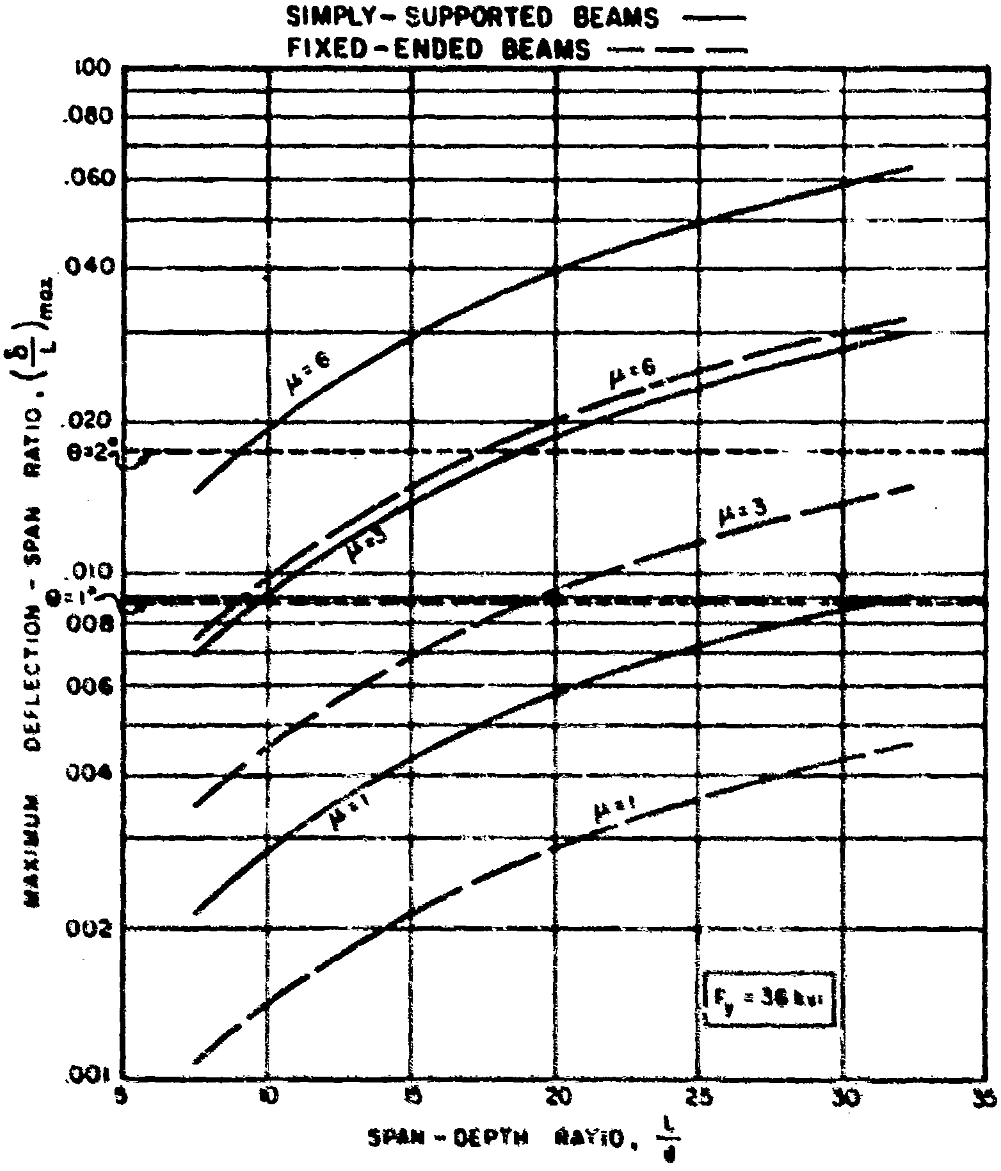
$\mu = 3$ Limit for reusable members

$\mu = 6$ Limit for non-reusable members

The limits for maximum rotation are also indicated in terms of the ratio $\delta/L$, the ratio $1/114 = 0.00877$ corresponding to $\theta_{max} = 1°$ (reusable), and $1/57 = 0.0175$ for $\theta_{max} = 2°$ (non-reusable).

The usual design range for the L/d ratio varies from 10 to 30, the latter conforming to the AISC Specification guideline that the maximum L/d ratio for beams should not exceed a value between 22 and 28 depending upon the end connections.

For a given design category (reusable or non-reusable) and particular support condition, the lower of the two applicable curves, either support rotation or ductility ratio will govern the design at a particular L/d value. Inspection of Figure 2.4(a), for $F_y = 36$ ksi, indicates that for simply-supported beams, the support rotation governs the design over the practical range of L/d for both design categories. In the case of fixed-end beams, the designs are limited by the ductility ratio criterion for L/d values less than 19 and 17.5 for the reusable and non-reusable categories, respectively. Qualitatively, the same observations hold for beams with $F_y = 50$ ksi as shown in Figure 2.4(b). As would be expected for this high-strength steel at the higher L/d ratios, the maximum support rotations of 1° and 2° restrict these beams to relatively lower ductility ratios as compared to the ductility ratios for beams with $F_y = 36$ ksi.



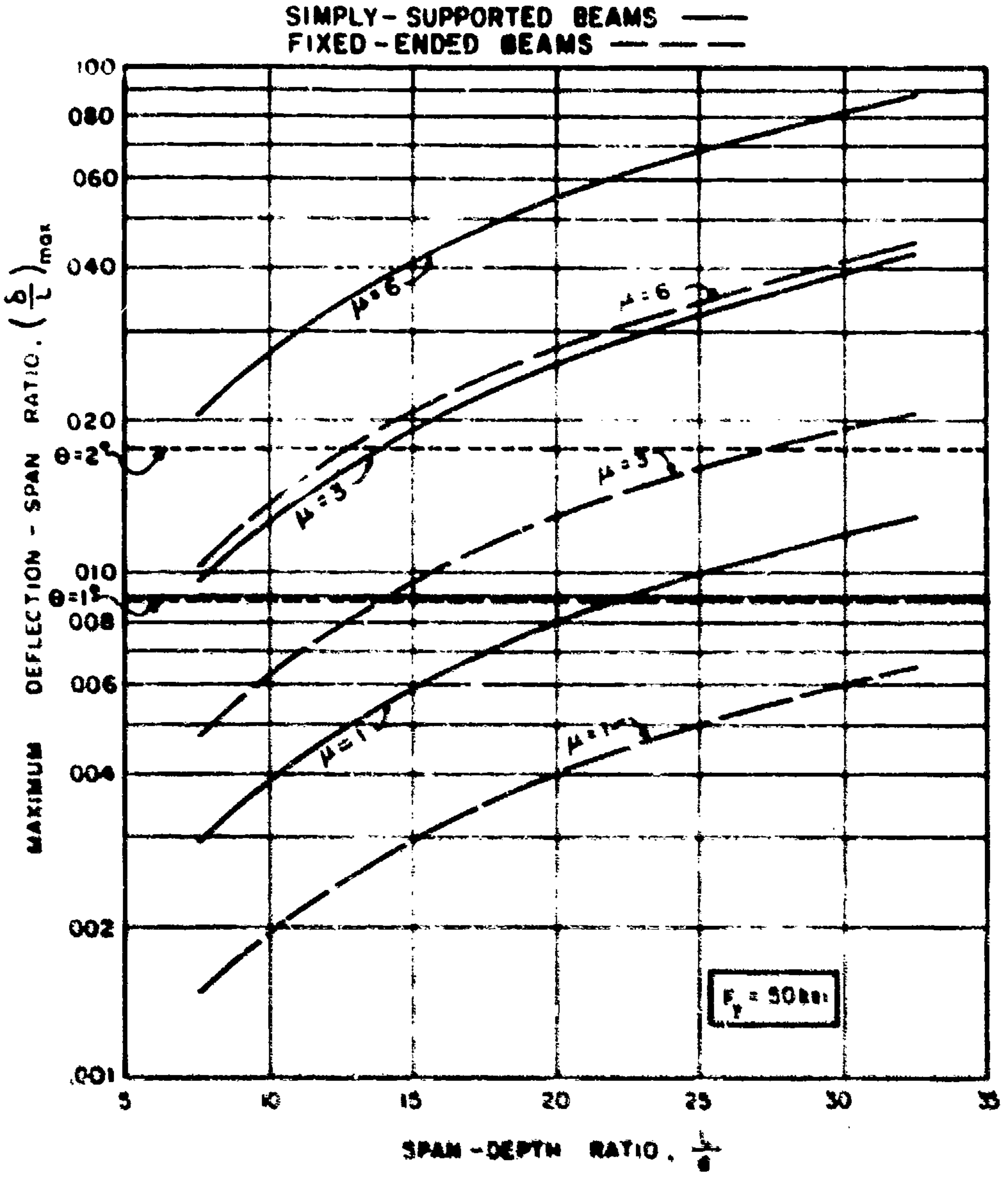
Reusable structures:

$\theta_{max} = 1°$ or $\mu_{max} = 3$, whichever governs

Non - reusable structures

$\theta_{max} = 2°$ or $\mu_{max} = 6$, whichever governs

Figure 2.4(a) Ductility ratio and maximum rotation criteria for compact I-and W-shaped sections in simple bending, $F_y$ = 36 ksi.



Reusable structures

$\theta_{max} = 1°$ or $\mu_{max} = 3$, whichever governs

Non-reusable structures:

$\theta_{max} = 2°$ or $\mu_{max} = 6$, whichever governs

Figure 2.4(b) Ductility ratio and maximum rotation criteria for compact I- and W-shaped sections in simple bending, $F_y = 50$ ksi.

In the following section, the deformation criteria are summarized for frames, beams and other structural elements including plates, cold-formed steel panels and open-web joists.

### 2.3.3 Summary of Deformation Criteria

(1) Beam elements including purlins, spandrels and girts

(a) Reusable structures

$\theta_{max} = 1^{o}$ or $\mu_{max} = 3$, whichever governs

(b) Non-reusable structures

$\theta_{max} = 2^{o}$ or $\mu_{max} = 6$, whichever governs

NOTE: For doubly-symmetric beams subject to bi-axial bending, more stringent criteria are recommended in Section 3.5.

(2) Frame structures

(a) Reusable structures

For sidesway, maximum $\delta/H = 1/50$

For individual frame members, $\theta_{max} = 1^{o}$

NOTE: For $F_y = 36$ ksi, $\theta_{max}$ should be reduced according to the following relationship for L/d less than 13,

$\theta_{max} = 0.07\ L/d + 0.09$

For the higher yield steels, $\theta_{max} = 1^{o}$ governs over the practical range of L/d values.

(b) Non-reusable structures

For sidesway, maximum $\delta/H = 1/25$

For individual frame members, $\theta_{max} = 2^{o}$

NOTE: For $F_y = 36$ ksi, $\theta_{max}$ should be reduced according to the following relationship for L/d less than 13,

$\theta_{max} = 0.14\ L/d + 0.18$

For the higher yield steels, $\theta_{max} = 2^{\circ}$ governs over the practical range of L/d values.

(3) Plates

(a) Reusable structures

$\theta_{max} = 2^{\circ}$ or $\mu_{max} = 5$, whichever governs

(b) Non-reusable structures

$\theta_{max} = 4^{\circ}$ or $\mu_{max} = 10$, whichever governs

(4) Cold-formed steel floor and wall panels

(a) Reusable structures

$\theta_{max} = 0.9^{\circ}$ or $\mu_{max} = 1.25$, whichever governs

(b) Non-reusable structures

$\theta_{max} = 1.8^{\circ}$ or $\mu_{max} = 1.75$, whichever governs

NOTE: A discussion of the behavior of cold-formed panels is presented in Section 3.7.

(5) Open-web joists

(a) Reusable structures

$\theta_{max} = 1^{\circ}$ or $\mu_{max} = 2$, whichever governs

(b) Non-reusable structures

$\theta_{max} = 2^{\circ}$ or $\mu_{max} = 4$, whichever governs

NOTE: For joists controlled by maximum end reaction, $\mu_{max}$ is limited to 1 for both reusable and non-reusable structures.

In the above:

$\theta_{max}$ = maximum member end rotation (degrees) measured from the chord joining the member ends.

$\delta$ = relative sidesway deflection between stories

H = story height

$\mu_{max}$ = maximum ductility ratio ($X_m/X_E$) for an element

L/d = span/depth ratio for a beam element

Finally, it should be recognized that the designer must be cognizant of any operational requirements which, under certain circumstances, may override the deformation criteria summarized above.

## 2.4 Secondary Modes of Failure

In the process of designing for the ductile mode of failure, it is important to follow certain provisions in order to avoid premature failure of the structure, i.e., to insure that the structure can develop its full plastic resistance.

These secondary modes of failure can be grouped in two main categories:

(1) Instability modes of failure

(2) Brittle modes of failure.

### 2.4.1 Instability Modes of Failure

In this category, the problem of structural instability at two levels is of concern:

(1) Overall buckling of the structural system as a whole

(2) Buckling of the component elements.

Overall buckling of framed structures can occur in two essentially different manners:

(1) The load and the structure are symmetric; deformations remain also symmetric up to a critical value of the load for which a sudden change in configuration will produce instant anti-symmetry, large sidesway displacement, and eventually a failure by collapse, if not by excessive deformations. This type of instability can also occur in the elastic domain, before substantial deformation or any plastification has taken place. It is called "instability by bifurcation".

(2) The loading or the structure or both are non-symmetric. With the application of the load, sidesway develops progressively. In such cases, the vertical loads acting through the sidesway displacements, commonly called "the P-Δ effect", create second order bending moments that magnify, in turn, the deformation. Because of rapidly increasing displacements, plastic hinges form, thereby decreasing the rigidity of the structure and causing more sidesway. This type of instability is related to a continuous deterioration of the stiffness leading to an early failure by either a collapse mechanism or excessive sidesway.

Frame instability need not be explicitly considered in the plastic design of one- and two-story unbraced frames provided that the individual columns and girders are designed according to the beam-column criteria of Chapter 4. For frames greater than two stories, bracing is normally required according to the AISC provisions for plastic design in order to insure the overall stability of the structure. However, if an inelastic dynamic frame analysis is performed to determine the complete time-history of the structural response to the blast loading, including the P-Δ effects, it may be established, in particular cases, that lateral bracing is not necessary in a frame greater than two stories. As mentioned previously, the computer program DYNFA may be employed for such an analysis.

Buckling of an element in the structure (e.g., a beam, girder or column) can occur under certain loading and end conditions. Instability is of two types:

(1) Buckling of the member as a whole, e.g., lateral torsional buckling.

(2) Local buckling at certain sections, including flange buckling and web crippling.

Provisions for plastic design of beams and columns are presented in Chapters 3 and 4.

### 2.4.2 Brittle Modes of Failure

Under dynamic loading, there is an enhanced possibility that brittle fracture can develop under certain conditions. Since this type of failure is sudden in nature and difficult to predict, it is very important to dimimish the risk of such premature failure.

The complexity of the brittle fracture phenomena precludes a complete quantitative definition. As a result, it is impossible to establish simple rules for design.

Brittle fractures are caused by a combination of adverse circumstances that may include a few, some, or all of the following:

(1) Local stress concentrations and residual stresses

(2) Poor welding

(3) The use of a notch sensitive steel

(4) Shock loading or rapid strain rate

(5) Low temperatures

(6) Decreased ductility due to strain aging

(7) The existence of a plane strain condition causing a state of triaxial tension stresses, especially in thick gusset plates, thick webs and in the vicinity of welds

(8) Non-uniformly-distributed blast loads.

The problem of brittle fracture is closely related to the detailing of connections, a topic that will be treated in a separate chapter of this report. However, there are certain general guidelines to follow in order to minimize the danger of brittle fracture:

(1) Steel material must be selected to conform with the condition anticipated in service.

(2) Fabrication and workmanship should meet high standards, e.g., sheared edges and notches should be avoided, and material that has been severely cold-worked should be removed.

(3) Proportioning and detailing of connections should be such that free movement of the base material is permitted, stress concentrations and triaxial stress conditions are avoided, and adequate ductility is provided.

# CHAPTER 3

# BEAMS AND PLATES

## 3.1 Introduction

The emphasis in this chapter is on the dynamic plastic design of structural steel beams and plates. Design data have been derived from the static provisions of the AISC Specification with necessary modifications and additions for blast design. Since the basic theory of plastic design of steel members is available in a number of the references to this report, this material is presented with a minimum of commentary. It should be noted that all provisions on plastic design in the AISC Specification apply, except as modified in this report.

The calculation of the dynamic flexural capacity of beams and plates is described in detail in Section 3.2. In Sections 3.3 and 3.4, the necessary information is presented for determining the equivalent bilinear resistance-deflection functions used in evaluating the basic flexural response of both beams and plates. Also presented are the supplementary considerations of adequate shear capacity and local and overall stability which are necessary for the process of hinge formation, moment redistribution and inelastic hinge rotation to proceed to the development of a full collapse mechanism.

In addition, design provisions for the following special topics are included in this report: unsymmetrical bending (Section 3.5), blast doors (Section 3.6), cold-formed steel floor and wall panels (Section 3.7) and open-web joists (Section 3.8).

## 3.2 Dynamic Flexural Capacity

The dynamic flexural capacity of a steel section is related to its static flexural capacity by the ratio of the dynamic to the static yield stresses of the material (see Chapter 2). Thus, the ultimate dynamic moment resisting capacity of a steel section is given by

$$M_{pu} = F_{dy}Z \qquad (3.1)$$

where $F_{dy}$ is the dynamic yield strength of the material and $Z$ is the plastic section modulus. For standard I-shaped sections (S, W and M shapes), the plastic section modulus is approximately 1.15 times the elastic modulus for strong axis bending and may be obtained from standard manuals on steel design. For plates or rectangular cross-section beams, the plastic section modulus is 1.5 times the elastic section modulus.

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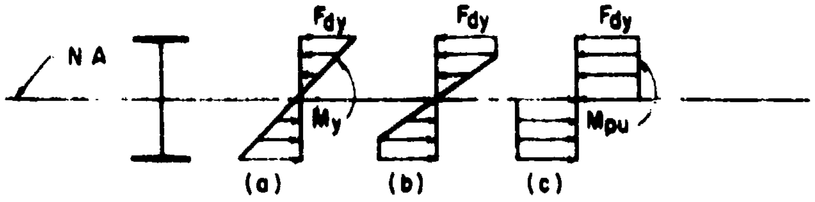
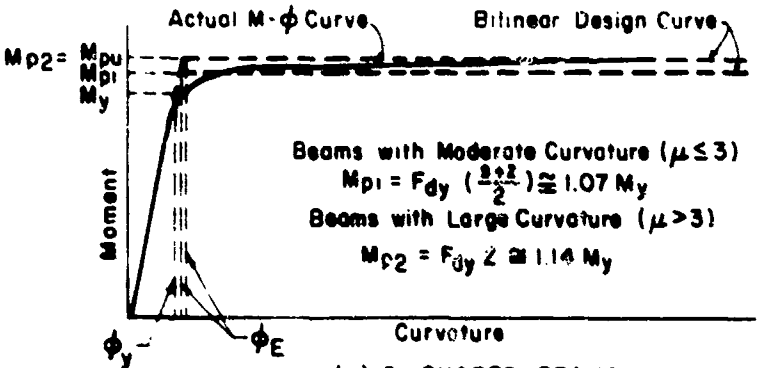
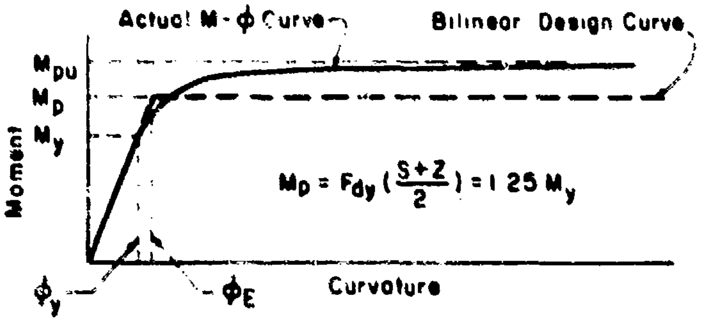


Figure 3.1 Theoretical stress distribution for pure bending at various stages of loading.



(a) I-SHAPED BEAMS



(b) PLATES AND RECTANGULAR BEAMS

Figure 3.2 Dynamic moment-curvature diagrams for simply-supported beams and plates.

It is generally assumed that a fully plastic section offers no additional resistance to load. Additional resistance due to strain hardening of the material is neglected because the strains necessary for this phenomenon to become significant cannot normally be tolerated. In blast design, although strains well into the strain-hardening range may be tolerated, the corresponding additional resistance is generally not sufficient to warrant analytical consideration.

Figure 3.1 shows the stress distribution at various stages of deformation for a plastic hinge section. Theoretically, the beam bends elastically until the outer fiber stress reaches $F_{dy}$ and the yield moment designated by $M_y$ is attained [Figure 3.1(a)]. As the moment increases above $M_y$, the yield stress progresses inward from the outer fibers of the section towards the neutral axis as shown in Figure 3.1(b). As the moment approaches the fully plastic moment $M_{pu}$, a rectangular stress distribution as shown in Figure 3.1(c) is approached. The ratio between the fully plastic moment to the yield moment is equal to the shape factor for the section, i.e., the ratio between the plastic and elastic section moduli.

Representative moment-curvature relationships for simply-supported steel beams and plates are shown in Figure 3.2 In each case, the behavior is elastic until a curvature $\phi_y$ corresponding to the yield moment $M_y$ is reached. With further increase in load, the curvature increases at a greater rate as the fully plastic moment value, $M_{pu}$, is approached. Following the attainment of $M_{pu}$, the curvature increases while the moment remains constant at $M_{pu}$.

For design purposes, a bilinear representation of the moment-curvature relationship is employed as shown by the dashed lines in Figures 3.2(a) and 3.2(b). The plastic moment capacity to be used in design, $M_p$, is assigned in recognition of the actual nature of the transition between yield moment, $M_y$, and the fully plastic moment, $M_{pu}$. Since a substantial amount of plastic deformation must occur before $M_{pu}$ is attained, a reduced value of the plastic design moment [$M_{p1}$ in Figure 3.2(a)] is defined for beams with moderate curvature (corresponding to design ductility ratios less than or equal to 3). For larger amounts of plastic deformation (design ductility ratios greater than 3), the full value of the plastic moment capacity ($M_{p2} = M_{pu}$) may be used.

For plates and rectangular cross-section beams [Figure 3.2(b)], $M_{pu}$ is 50 percent greater than $M_y$ and the nature of the transition from yield to the fully plastic condition depends upon the plate geometry and end conditions. It is presently recommended that a capacity midway between $M_y$ and $M_{pu}$ be used to define the plastic design moment, $M_p$, in all cases independent of ductility ratio. The equivalent elastic curvature, $\phi_E$, corresponds to the development of the plastic moment capacity.

To summarize, the equivalent plastic moment shall be computed as follows:

1. For beams with design ductility ratios less than or equal to 3 and for plates and rectangular cross-section beams with any design ductility ratio:

$$M_p = F_{dy}(S + Z)/2^* \qquad (3.2)$$

where S and Z are the elastic and plastic section moduli, respectively.

2. For beams with design ductility ratios greater than 3 and beam columns:

$$M_p = F_{dy}Z \qquad (3.3)$$

Equation 3.2 is consistent with test results for beams with moderate curvatures and for plates. For beams which are allowed to undergo large curvatures, Equation 3.3, based upon full plastification of the section, is considered reasonable for design purposes.

It is important to note that the above pertains to beams or plates which are supported against buckling. Design provisions for guarding against local and overall buckling of beams during plastic deformation are discussed in Sections 3.3.4, 3.3.5 and 3.3.6.

In the analysis of structural steel members, it is assumed that the plastic hinge formation is concentrated at a section. Actually, the plastic region extends over a certain length that depends on the type of loading (concentrated or distributed) and on the shape factor of the cross-section. The extent of the plastic hinge has no substantial influence on the ultimate capacity; it has, however, an influence on the final magnitude of the deflection. For all practical purposes, the assumption of a concentrated plastic hinge is adequate.

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*See Section 3.4.3 for a discussion of the effect of shear on the available moment capacity for plates.

## 3.3 Design of Beams and Continuous Beams

### 3.3.1 Resistance Functions

The single-degree-of-freedom analysis which serves as the basis for the flexural response calculation requires that the equivalent stiffness and ultimate resistance be defined for both beam elements and continuous beams.

Formulas for determining the stiffness and resistance for one-way steel beam elements are presented in Table 3.1. The ultimate resistance values correspond to developing a full collapse mechanism in each case. The equivalent stiffnesses correspond to load-deflection relationships that have been idealized as bilinear functions with initial slopes so defined that the areas under the idealized load deflection diagrams are equal to the areas under the actual diagrams at the point of inception of fully plastic behavior of the beam.

It is important to note that the data and discussions in TM 5-1300 on partial failure for concrete do not apply to steel where the fully plastic resistance is taken constant with an increase in deflection of the beam.

The beam relationships for defining the bilinear resistance function for multi-span continuous beams under uniform loading are summarized below. These expressions are predicated upon the formation of a three-hinge mechanism in each span. Maximum economy normally dictates that the span lengths and/or member sizes be adjusted such that a mechanism forms simultaneously in all spans.

It must be noted that the development of a mechanism in a particular span of a continuous beam assumes compatible stiffness properties at the end supports. If the ratio of the length of the adjacent spans to the span being considered is excessive (say, greater than 3), it may not be possible to reach the limit load without the beam failing by excessive deflection.

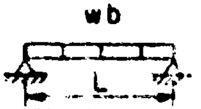
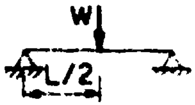
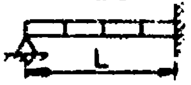
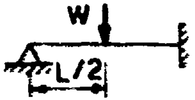
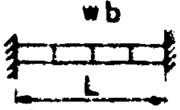
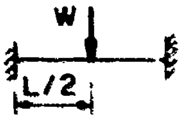
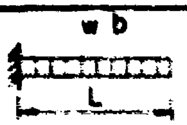
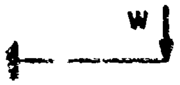
For uniformly-distributed loading on equal spans or spans which do not differ in length by more than 20 percent, the following relationships can be used to define the bilinear resistance function:

Two-span continuous beam:

$$R_u = r_u bL = 12\ M_p/L \qquad (3.4)$$

$$K_E = 163\ EI/L^3 \qquad (3.5)$$

## TABLE 3.1 ULTIMATE RESISTANCE AND STIFFNESS OF BEAM ELEMENTS.

| Member And Load Configuration | Ultimate Flexural Resistance | Equivalent Elastic Stiffness |
|---|---|---|
|  | $r_u bL = 8.0 \frac{M_p}{L}$ | $K_E = \frac{384EI}{5L^3}$ |
|  | $R_u = 4.0 \frac{M_p}{L}$ | $K_E = \frac{48EI}{L^3}$ |
|  | $r_u bL = 12.0 \frac{M_p}{L}$ | $K_E = \frac{160EI}{L^3}$ |
|  | $R_u = 6.0 \frac{M_p}{L}$ | $K_E = \frac{106EI}{L^3}$ |
|  | $r_u bL = 16.0 \frac{M_p}{L}$ | $K_E = \frac{307EI}{L^3}$ |
|  | $R_u = 8.0 \frac{M_p}{L}$ | $K_E = \frac{192EI}{L^3}$ |
|  | $r_u bL = 2.0 \frac{M_p}{L}$ | $K_E = \frac{8EI}{L^3}$ |
|  | $R_u = \frac{M_p}{L}$ | $K_E = \frac{3EI}{L^3}$ |

Where:

$b$ = Width Of Contributory Loading Area

$M_p$ = Plastic Moment Capacity

$r_u$ = Ultimate Resistance per Unit Area

$R_u$ = Ultimate Total Resistance

$w$ = Load per Unit Area

$W$ = Total Concentrated Load

Exterior span of continuous beams with 3 or more spans:

$$R_u = r_u bL = 11.7M_p/L \qquad (3.6)$$

$$K_E = 143EI/L^3 \qquad (3.7)$$

Interior span of continuous beam with 3 or more spans:

$$R_u = r_u bL = 16.0M_p/L \qquad (3.8)$$

$$K_E = 300EI/L^3 \qquad (3.9)$$

For design situations which do not meet the required conditions, the bilinear resistance function may be developed by application of the basic procedures of plastic analysis.

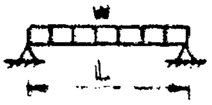
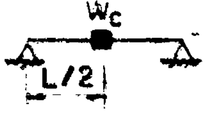
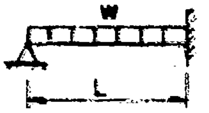
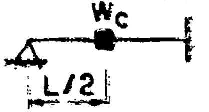
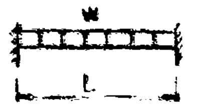
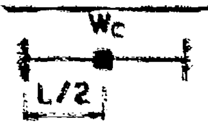
### 3.3.2 Design for Flexure

The design of a structure to resist the blast of an accidental explosion consists essentially of the determination of the structural resistance required to limit calculated deflections to within the prescribed maximum values (Section 2.3.3). In general, the resistance and deflection may be computed on the basis of flexure provided that the shear capacity of the web is not exceeded. Elastic shearing deformations of steel members are negligible as long as the depth to span ratio is less than about 0.2 and hence, a flexural analysis is normally sufficient for establishing maximum deflections.

As previously discussed, steel structures designed to resist accidental explosions will generally respond either to the pressure only or to the pressure-time relationship when they are situated in the low pressure and intermediate pressure design ranges. Steel structures in a high pressure design range would tend to respond more to the impulse produced by the blast pressures. For each case, the appropriate dynamic response charts and equations for computing the required resistance of the member in terms of the limiting deflection or ductility ratio are given in Chapter 6, Sections 6.3 through 6.9, of TM 5-1300. In addition, Figure 3.3 presents the dynamic load factor and time-to-maximum response charts for linearly elastic systems.

To supplement the data in Chapter 6 of TM 5-1300, Table 3.2 gives natural periods of vibration of steel beams for several support conditions. The following expression can be used to determine the natural period of vibration for any system for which the total effective mass and equivalent elastic stiffness is known:

## TABLE 3.2 EFFECTIVE NATURAL PERIOD OF VIBRATION FOR ELASTIC BEAM ELEMENTS.

| Member | Period |
|---|---|
|  | $T_N = 0.64\, L^2 \sqrt{\frac{w}{gEI}}$ |
|  | $T_N = 0.91 \sqrt{\frac{W_c}{g}\,\frac{L^3}{EI}}$ |
|  | $T_N = 0.42\, L^2 \sqrt{\frac{w}{gEI}}$ |
|  | $T_N = 0.61 \sqrt{\frac{W_c}{g}\,\frac{L^3}{EI}}$ |
|  | $T_N = 0.28\, L^2 \sqrt{\frac{w}{gEI}}$ |
|  | $T_N = 0.45 \sqrt{\frac{W_c}{g}\,\frac{L^3}{EI}}$ |

**Where:**

$w$ = Weight per unit length (lb/ft)

$W_c$ = Total concentrated weight (lb)

$L$ = Span length (ft)

$g$ = Acceleration due to gravity (32.2 ft/sec$^2$)

$E$ = Modulus of elasticity (psf)

$I$ = Moment of inertia (ft$^4$)

$T_N$ = Effective natural period (sec)

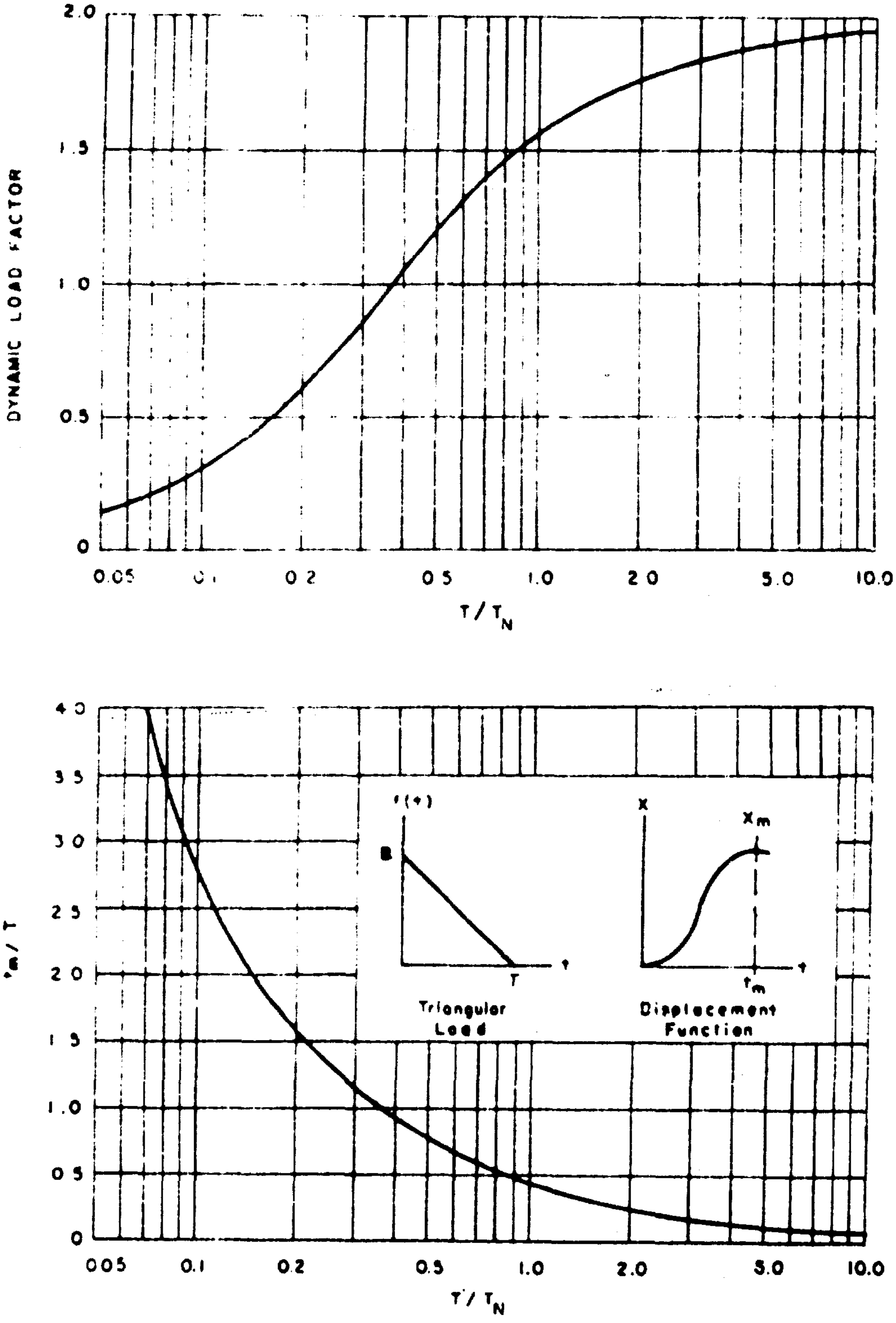


Figure 3.3 Dynamic load factor and $t_m/T$ curves for linearly elastic system, triangular load.

$$T_N = 2\pi\sqrt{M_e/K_E} \qquad (3.10)$$

where $M_e$ = total effective mass

$K_E = R_u/X_E$

$R_u$ = ultimate total resistance

$X_E$ = equivalent elastic deflection

For preliminary design of beams and plates situated in low or intermediate pressure ranges, it is suggested that the structure be designed to have an equivalent static ultimate resistance equal to 1.0 and 0.8 times the peak blast force for reusable and non-reusable structures, respectively. Since the duration of the loading for low to intermediate pressure ranges will generally be the same or longer than the period of vibration of the structure and the ductility factor is usually at least 3.0, revisions to this preliminary design from a dynamic analysis will usually not be substantial. However, for structures where the loading environment pressure is such that the load duration is short as compared with the period of vibration of the structure, this procedure may result in a substantial overestimate of the required resistance.

The rebound behavior of the structure must not be overlooked. Procedures and data for calculating the elastic rebound of structures are contained in Chapter 6 of TM 5-1300. The provisions of Section 3.3.4, Local Buckling, and Section 3.3.6, Lateral Bracing, shall apply in the design for rebound.

### 3.3.3 Design for Shear

Shearing forces are of significance in plastic design primarily because of their possible influence on the plastic moment capacity of a steel member. At points where large bending moments and shear forces exist, the assumption of an ideal elasto-plastic stress-strain relationship indicates that during the progressive formation of a plastic hinge, there is a reduction of the web area available for shear. This reduced area could result in an initiation of shear yielding and possibly reduce the moment capacity.

However, it has been found experimentally that I-shaped sections achieve their fully plastic moment capacity provided that the average shear stress over the full web area is less than the yield stress in shear. This result can basically be attributed to the fact that I-shaped sections carry moment predominantly through the flanges and shear predominantly through the web. Other contributing factors include the beneficial effects of strain hardening and the fact that combinations of high shear and high moment generally occur at locations where the moment gradient is steep.

The yield capacity of steel beams in shear is given by:

$$V_p = F_{dv}A_w \qquad (3.11)$$

where $V_p$ is the shear capacity, $F_{dv}$ is the dynamic shear yield strength of the steel (Section 2.2.3) and $A_w$ is the area of the web. For I-shaped beams and similar flexural members with thin webs, only the web area between flange plates should be used in calculating $A_w$.

For several particular load and support conditions, equations for the support shears, V, for one-way elements are given in Table 3.3. As discussed above, as long as the acting shear V does not exceed $V_p$, I-shaped sections can be considered capable of achieving their full plastic moment. If V is greater than $V_p$, the web area of the chosen section is inadequate and either the web must be strengthened or a different section should be selected.

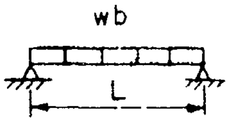
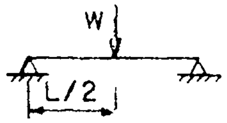
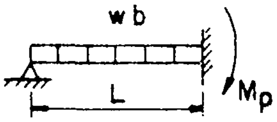
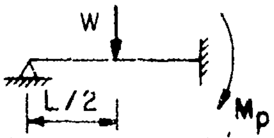
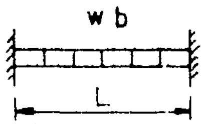
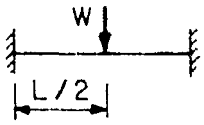
However, for cases where the web is being relied upon to carry a significant portion of the moment capacity of the section, such as rectangular cross-section beams or built-up sections, the influence of shear on the available moment capacity must be considered as treated below in Section 3.4.3.

In order to avoid plastic shear deformations in open web joists and similar trussed structures, the computed maximum tensile stress in the web members should not exceed 90 percent of the dynamic yield stress of the material. Web members in compression must meet the requirements of Chapter 4. Design procedures for open-web joists are summarized in Section 3.8.

### 3.3.4 Local Buckling

In order to insure that a steel beam will attain fully plastic behavior and possess the assumed ductility at plastic hinge locations, it is necessary that the elements of the beam section meet minimum thickness requirements sufficient to prevent a local buckling failure. Adopting the plastic design requirements of the AISC Specification, the width-thickness ratio for flanges of rolled I- and W-shapes and similar built-up single web shapes that would be subjected to compression involving plastic hinge rotation shall not exceed the following values:

TABLE 3.3 SUPPORT SHEAR FOR BEAM ELEMENTS.

| Member And Load Configuration | Maximum Support Shear |
|---|---|
|  | $V = \frac{r_u bL}{2}$ |
|  | $V = \frac{R_u}{2}$ |
|  | $V = \frac{r_u bL}{2} + \frac{M_p}{L}$ |
|  | $V = \frac{R_u}{2} + \frac{M_p}{L}$ |
|  | $V = \frac{r_u bL}{2}$ |
|  | $V = \frac{R_u}{2}$ |

Where:

$b$ = Width Of Loaded Area
$V$ = Support Shear
$r_u$ = Ultimate Resistance per Unit Area
$R_u$ = Ultimate Total Resistance
$w$ = Load per Unit Area
$W$ = Total Concentrated Load

| $F_y$(ksi) | $b_f/2t_f$ |
|---|---|
| 36 | 8.5 |
| 42 | 8.0 |
| 45 | 7.4 |
| 50 | 7.0 |
| 55 | 6.6 |
| 60 | 6.3 |
| 65 | 6.0 |

where $F_y$ is the specified minimum static yield stress for the steel, $b_f$ is the flange width and $t_f$ is the flange thickness.

The width-thickness ratio of similarly compressed flange plates in box sections and cover plates shall not exceed $190/\sqrt{F_y}$. For this purpose, the width of a cover plate shall be taken as the distance between longitudinal lines of connecting rivets, high-strength bolts or welds.

The depth-thickness ratio of webs of members subjected to plastic bending shall not exceed the value given by Equation 3.12 or 3.13, as applicable.

$$\frac{d}{t_w} = \frac{412}{\sqrt{F_y}}\left(1 - \frac{1.4P}{P_y}\right) \text{ when } \frac{P}{P_y} \leq 0.27 \qquad (3.12)$$

$$\frac{d}{t_w} = \frac{257}{\sqrt{F_y}} \text{ when } \frac{P}{P_y} > 0.27 \qquad (3.13)$$

where $P$ = the applied compressive load

$P_y$ = the plastic axial load equal to the cross-sectional area times the specified minimum static yield stress $F_y$

These equations for local buckling under dynamic loading have been adopted from the AISC provisions for static loading. However, since the actual process of buckling takes a finite period of time, the member must accelerate laterally and the mass of the member provides an inertial force retarding this acceleration. For this reason, loads that might otherwise cause failure may be applied to the members for very short durations if they are removed before the buckling has occurred. Hence, it is appropriate and conservative to apply the criteria developed for static loads to the case of dynamic loading of relatively short duration.

These requirements on cross-section geometry should be adhered to in the design of all members for blast loading. However, in the event that it is necessary to evaluate the load-carrying capacity of an existing structural member which does not meet these provisions, the ultimate capacity should be reduced in accordance with the recommendations made in the Commentary and Appendix C of the AISC Specification.

### 3.3.5 Web Crippling

Since concentrated loads and reactions along a short length of flange are carried by compressive stresses in the web of the supporting member, local yielding may occur followed by crippling or crumpling of the web. Stiffeners bearing against the flanges at load points and fastened to the web are usually employed in such situations to provide a gradual transfer of these forces to the web.

Provisions for web stiffeners, as given in Section 1.15.5 of the AISC Specification, should be used in dynamic design. In applying these provisions, $F_y$ should be taken equal to the specified static yield strength of the steel.

### 3.3.6 Lateral Bracing

Members subjected to bending about their strong axis may be susceptible to lateral-torsional buckling in the direction of the weak axis if their compression flange is not laterally braced. Therefore, in order for a plastically-designed member to reach its collapse mechanism, lateral supports must be provided at the plastic hinge locations and at a certain distance from the hinge location. The distance from the brace at the hinge location to the adjacent braced points should not be greater than $l_{cr}$ as determined from either Equation 3.14 or 3.15, as applicable:

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_{dy}} + 25 \quad \text{when } +1.0 \geq \frac{M}{M_p} > -0.5 \qquad (3.14)$$

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_{dy}} \quad \text{when } -0.5 \geq \frac{M}{M_p} \geq -1.0 \qquad (3.15)$$

where $r_y$ = the radius of gyration of the member about its weak axis

$M$ = the lesser of the moments at the ends of the unbraced segment

$\frac{M}{M_p}$ = the end moment ratio, is positive when the segment is bent in reverse curvature and negative when bent in single curvature

Since the last hinge to form in the collapse mechanism is required to undergo less plastic deformation, the bracing requirements are somewhat less stringent. For this case, in order to develop the full plastic design moment, $M_p$, the following relationship may be used for members with design ductility ratios less than or equal to 3.

$$l/r_T = \sqrt{(102\text{x}10^3 C_b)/F_y} \qquad (3.16)$$

where $l$ = distance between cross-sections braced against twist or lateral displacement of the compression flange

$r_T$ = radius of gyration of a section comprising the compression flange plus one-third of the compression web area taken about an axis in the plane of the web

$C_b$ = bending coefficient defined in Section 1.5.1.4.6a of the AISC Specification

However, in structures designed for ductility ratios greater than 3, the bracing requirements of Equations 3.14 and 3.15 must be met.

The bracing requirements for non-yielded segments of members and the bracing requirements for members in rebound can be determined from the following relationship:

$$F = 1.67\left[2/3 - \frac{F_y(l/r_T)^2}{1530\text{x}10^3 C_b}\right]F_y \qquad (3.17)$$

where $F$ = the maximum bending stress in the member, and in no case greater than $F_y$

When $F$ equals $F_y$, this equation reduces to the $l/r_T$ requirement of Equation 3.16.

Lateral bracing support is often provided by floor beams, joists or purlins which frame into the member to be braced. Hence, in design, the unbraced lengths are essentially fixed by the spacing of the purlins and girts and this given spacing must be checked for lateral buckling. Since this spacing is usually uniform, the particular unbraced length that must be investigated will be the one with the largest moment ratio. For the same reason, the spacing of bracing adjacent to the last hinge and in the non-yielding segments of a member should be checked when the spacing in these portions exceeds $l_{cr}$.

When the compression flange is securely connected to steel decking or siding, this will constitute adequate lateral bracing in most cases. In addition, inflection points (points of contraflexure) can be considered as braced points.

Members built into a masonry wall and having their web perpendicular to this wall can be assumed to be laterally supported with respect to their weak axes of bending. In addition, points of contraflexure can be considered as braced points, if necessary.

In order to function adequately, the bracing member must meet certain minimum requirements on axial strength and axial stiffness (ASCE-WRC Commentary on Plastic Design). These requirements are quite minimal in relation to the properties of typical framing members.

Lateral braces should be welded or securely bolted to the compression flange and, in addition, a vertical stiffener should generally be provided at bracing points where concentrated vertical loads are also being transferred. Plastic hinge locations within uniformly loaded spans do not generally require a stiffener.

## 3.4 Design of Plates

### 3.4.1 Resistance Functions

Stiffness and resistance factors for one- and two-way plate elements are defined in Chapter 5, Sections III and IV of TM 5-1300 and by the supplementary charts in Appendix A. These factors, originally developed for concrete elements and based upon elastic deflection theory and the yield-line method, are also appropriate for defining the stiffness and ultimate load-carrying capacity of ductile structural steel plates. In applying these factors to steel plates, the modulus of elasticity should be taken equal to 29,000,000 psi. For two-way isotropic steel plates, the ultimate unit positive and negative moments are equal in all directions; i.e.,

$$M_{vn} = M_{vp} = M_{hn} = M_{hp} = M_p$$

where $M_p$ is defined by Equation 3.2. Since the stiffness factors were derived for plates with equal stiffness properties in each direction, they are not applicable to the case of orthotropic steel plates, such as stiffened plates, which have different stiffness properties in each direction.

### 3.4.2 Design for Flexure

The flexural design of a steel plate proceeds in essentially the same manner as the design of a beam. As for beams, it is suggested that dynamic load factors of 1.0 and 0.8 be used for the preliminary design of reusable and non-reusable plate structures, respectively. With the stiffness and resistance factors from Section 3.4.1 and taking into account the influence of shear on the available plate moment capacity as defined in Section 3.4.3, the dynamic response and rebound for a given blast loading may be determined from the response charts in Chapter 6 of TM 5-1300 and Figure 3.3. The recommended maximum deflections and support rotations for plates are provided in Section 2.3.3.

### 3.4.3 Design for Shear

In the design of rectangular plates, the effect of simultaneous high moment and high shear at negative yield lines upon the plastic strength of the plate may be significant. In such cases, the following interaction formula describes the effect of the support shear, V, upon the available moment capacity, M:

$$M/M_p = 1 - (V/V_p)^4 \qquad (3.18)$$

where $M_p$ is the fully plastic moment capacity in the absence of shear calculated from Equation 3.3 and $V_p$ is the ultimate shear capacity in the absence of bending determined from Equation 3.11 where the web area, $A_w$, is taken equal to the total cross-sectional area at the support.

For two-way elements, values for the ultimate support shears are presented in Chapter 5, Table 5-14, Section V of TM 5-1300. These shears may also be used for steel plates. However, the ultimate shearing stresses given in Section V for concrete elements are not applicable to steel.

It should be noted that due to the inter-relationship between the support shear, V, the unit ultimate flexural resistance,

$r_u$, of the two-way element, and the fully plastic moment resistance, $M_p$, the determination of the resistance of steel plates considering Equation 3.18, is not a simple calculation. Fortunately, however, the number of instances when negative yield lines with support shears are encountered for steel plates will be limited. Moreover, in most applications, the $V/V_p$ ratio is such that the available moment capacity is at least equal to the plastic design moment for plates (Equation 3.2).

To summarize, if the $V/V_p$ ratio on negative yield lines is less than 0.67, the plastic design moment for plates, as determined from Equation 3.2, should be used in design. However, if $V/V_p$ is greater than 0.67, the influence of shear on the available moment capacity must be accounted for by means of Equation 3.18.

## 3.5 Special Provisions for Unsymmetrical Bending

The term "unsymmetrical bending" refers to a situation where flexural members are subjected to transverse loads acting in a plane other than a principal plane. With this type of loading:

1. The member's neutral axis is not perpendicular to the plane of loading.
2. Stresses cannot in general be calculated by means of the simple bending formula (Mc/I).
3. The bending deflection does not coincide with the plane of loading but is perpendicular to the inclined neutral axis.
4. If the plane of loads does not pass through the shear center of the cross-section, bending is also accompanied by twisting.

Doubly-symmetric S-, W- and box sections acting as individual beam elements and subjected to bi-axial bending, i.e., unsymmetrical bending without torsion, can be treated in the following manner. The inclination of the elastic and plastic neutral axis through the centroid of the section can be calculated directly from the following relationship (see Fig. 3.4):

$$\tan \alpha = (I_x/I_y)\tan \phi \qquad (3.19)$$

where $\alpha$ = angle between the horizontal principal plane and the neutral axis

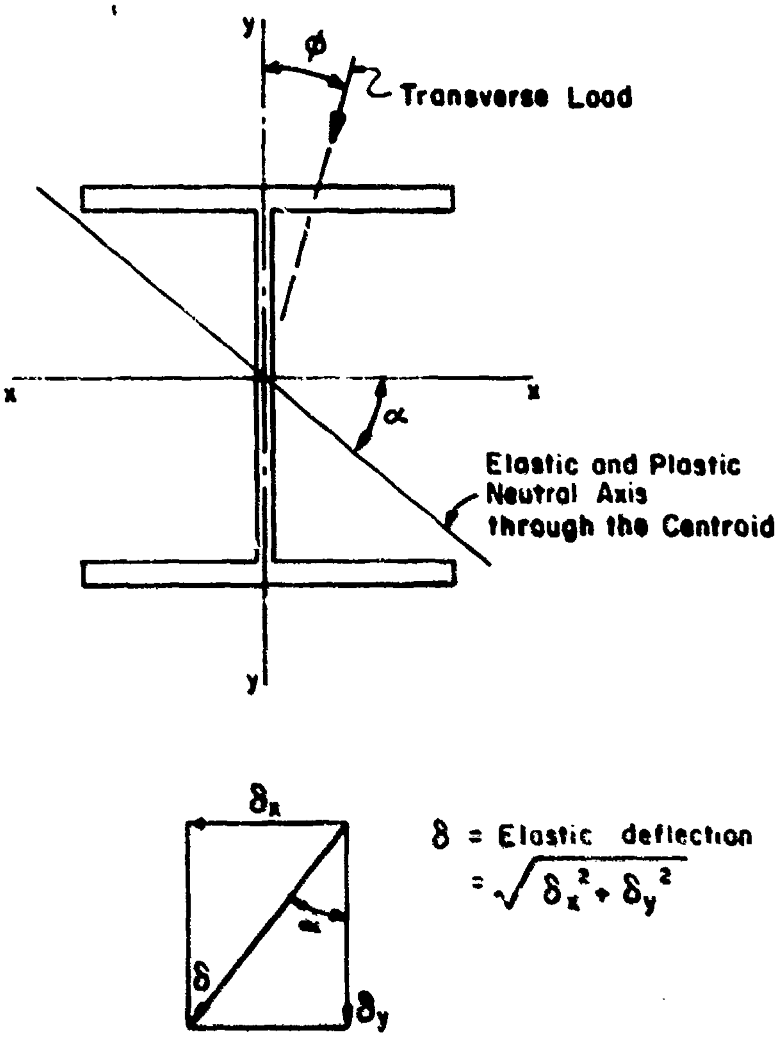

y
φ
Transverse Load
x
x
α
Elastic and Plastic
Neutral Axis
through the Centroid
y
$\delta_x$
α
δ
$\delta_y$
δ = Elastic deflection
$= \sqrt{\delta_x^2 + \delta_y^2}$

Figure 3.4 Biaxial bending of a doubly-symmetric section.

$\phi$ = angle between the plane of the load
and the vertical principal plane

and x and y refer to the horizontal and vertical principal axes of the cross-section.

The equivalent elastic section modulus may be evaluated from the following equation:

$$S = (S_xS_y)/(S_y\cos\phi + S_x\sin\phi) \qquad [3.20(a)]$$

where $S_x$ = elastic section modulus about the x-axis

$S_y$ = elastic section modulus about the y-axis.

The plastic section modulus can be calculated as the sum of the static moments of the fully yielded elements of the equal cross-section areas above and below the neutral axis, i.e.:

$$Z = A_cm_1 + A_tm_2 \qquad [3.20(b)]$$

NOTE: $A_cm_1 = A_tm_2$ for a doubly-symmetric section

where $A_c$ = area of cross-section in compression

$A_t$ = area of cross-section in tension

$m_1$ = distance from neutral axis to the centroid of the area in compression

$m_2$ = distance from neutral axis to the centroid of the area in tension

With these values of the elastic and plastic section moduli, the design plastic moment capacity can be determined from Equation 3.2.

In order to define the stiffness and bilinear resistance function, it is necessary to determine the elastic deflection of the beam. This deflection may be calculated by resolving the load into components acting in the principal planes of the cross-section. The elastic deflection, $\delta$, is calculated as the resultant of the deflections determined by simple bending calculations in each direction (see Fig. 3.4). The equivalent elastic deflection on the bilinear resistance function, $X_E$, may then be determined by assuming that the elastic stiffness is valid up to the development of the design plastic moment capacity, $M_p$.

The bracing requirements of Section 3.3.6 may not be totally adequate to permit a biaxially-loaded section to deflect into the inelastic range without premature failure. For lack of data, the provisions of Section 3.3.4 on lateral bracing may be used but the ductility ratios should be limited to 1.5 and 3.0 for the reusable and non-reusable design categories, respectively. In addition, as in Section 2.3, the total member end rotation corresponding to the total deflection due to the inclined load must be limited to $1^{o}$ and $2^{o}$ for reusable and non-reusable structures. The actual details of support conditions and bracing provided to such members by the other primary and secondary members of the frame must be carefully considered to ensure that the proper conditions exist to permit deflections in the inelastic range.

The inelastic behavior of sections subjected to unsymmetrical bending, with twisting, is not properly known at present. Consequently, the use of sections with the resultant load not acting through the shear center is not recommended in plastic design of blast-resistant structures, unless the sections are torsionally constrained. In actual installations, however, the torsional constraint offered to a purlin or girt by the flexural rigidity of the floor, roof or wall panels to which it is attached may force the secondary member to deflect in the plane of loading with little or no torsional effects. Under such conditions or when some other means of bracing is provided to prevent torsional rotation in both the loading and rebound phases of the response, such unsymmetrically loaded members may be capable of performing well in the plastic range. However, because of the limited data presently available, there is insufficient basis for providing practical design guidelines in this area. Hence, if a case involving unsymmetrical bending with torsion cannot be avoided in design, the maximum ductility ratio should be limited to 1.0.

For unconstrained torsion, a ductility ratio of 1.0 does not necessarily provide assurance of limited deformation. Unless special precautions are taken to restrict the torsional-flexural distortions that can develop under unsymmetrical loading with torsion, the flexural capacity of the member may be significantly reduced.

Fortunately, in blast design, the number of situations where unsymmetrical bending might cause difficulty is rather limited and even where encountered, it can be treated without serious economic penalty. Due to the fact that blast overpressure loads act normal to the surfaces of a structure, the use of doubly-symmetric cross-sections for purlins and girts (e.g., hot-rolled S-and W-sections or cold-formed channels used back-to-back) is generally recommended. In such cases, the deformation criteria for flexural members in Section 2.3.3 apply. For other applications such as built-up blast doors (Section 3.6), it is often desirable to use single-channel sections. In this case, the necessary torsional restraint is actually provided by the steel plates attached to one or both flanges. This fact, combined with the short spans involved, permits these sections to perform well in the inelastic range.

The treatment of the problem of biaxial bending in the presence of axial force which can arise in the design of rigid frames for a quartering load on a rectangular building is discussed in Chapter 5. Interaction equations for biaxial bending are presented in Chapter 4.

## 3.6 Special Provisions for Blast Doors

Blast doors can be grouped into two main categories, i.e., pressure doors and fragment doors, and may also be listed as to their method of opening:

1. Single-leaf

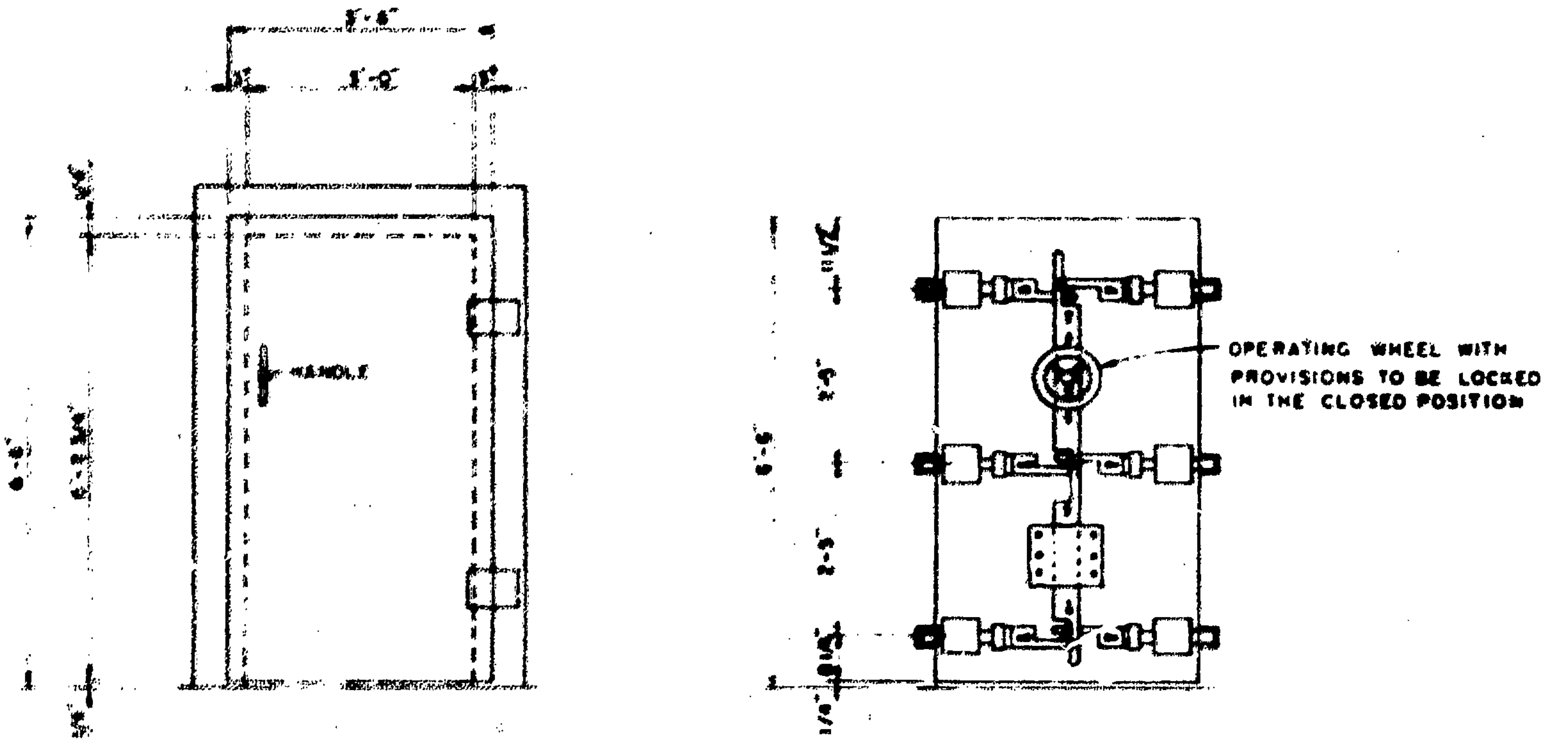
2. Double-leaf

3. Vertical lift

4. Horizontal sliding

The first category, pressure doors, includes doors which must resist blast pressures only. The second category, fragment doors, includes doors which must resist primary fragment impact in addition to blast pressures. Blast door exposure to fragment impact is a function of the door orientation to the source of an explosion, the distance from the explosion and the nature of the donor source. Procedures for defining fragment characteristics and for determining fragment input effects on concrete barriers are given in Chapters 4 and 8, respectively, of TM 5-1300. Information on penetration into steel plates is under development for Picatinny Arsenal by Ammann & Whitney and will be presented in the report "Primary Fragment Characteristics and Penetration of Steel, Concrete and Other Materials", Manufacturing Technology Directorate, Picatinny Arsenal.

There are two types of blast-door construction generally used. The first type is a heavy steel plate while the second is a built-up door consisting of a relatively light outside plate supported by a channel frame. The choice of either a plate door or a built-up door must be based upon comparison of their relative economy considering the particular blast pressure and fragment


3'-6"
3'-0"
HANDLE
6'-6"
OPERATING WHEEL WITH PROVISIONS TO BE LOCKED IN THE CLOSED POSITION

EXTERIOR

INTERIOR

Figure 3.5 Steel plate blast door.

environment along with any other factors such as insulation or hardware requirements. Plate doors are used for both high (50 psi and greater) and low pressure ranges and where fragment impact is critical. For lower pressure ranges (10 psi and less) where fragment impact is not a problem, the use of built-up doors is a practical arrangement.

Figure 3.5 illustrates a typical plate blast door. The direct load produced by the blast will be transmitted from the door to the supports by bearing; while reversal action of the door and the effects of negative pressure are transmitted to the door supports by several reversal bolts along the vertical edges. The reversal bolts eliminate the need to design the hinges for rebound. On wider doors, reversal bolts may be placed on the top and bottom door edges also, to take advantage of possible two-way action of the plate. Additional blast door details are provided in Appendix C.

Figure 3.6 depicts a typical built-up door. The peripheral door frame is composed of channels with other horizontal channels serving as intermediate supports for the door plate. All of the channel sections and the plate are connected by welding. The mechanism for reversal loads is similar to that used for the plate doors, except that the hinges usually are designed to serve as the reversal bolts on one side of the door due to the lower magnitude of the blast pressures involved.

The typical plate blast door is first sized for fragment penetration, if any, and then is designed to resist the blast pressures as a two-way plate with all sides simply-supported. Design charts, equations and procedures are given in Chapter 5 of TM 5-1300. Recommended design stresses and deformation limits are listed in Chapter 2 of this report. The reversal bolts are designed for the maximum rebound forces as determined from Fig. 6-8 of TM 5-1300.

For the built-up blast door, the exterior plate is designed as a continuous member supported by the transverse channels. In turn, e channels are then designed as simply-supported members, the applied load being equal to the blast pressures on the exterior plate. The vertical supporting channels are designed for the rebound forces as are the hinges and reversal bolts. If a backing plate is used, the plate should be designed for the maximum acceleration forces caused by the applied blast pressures.

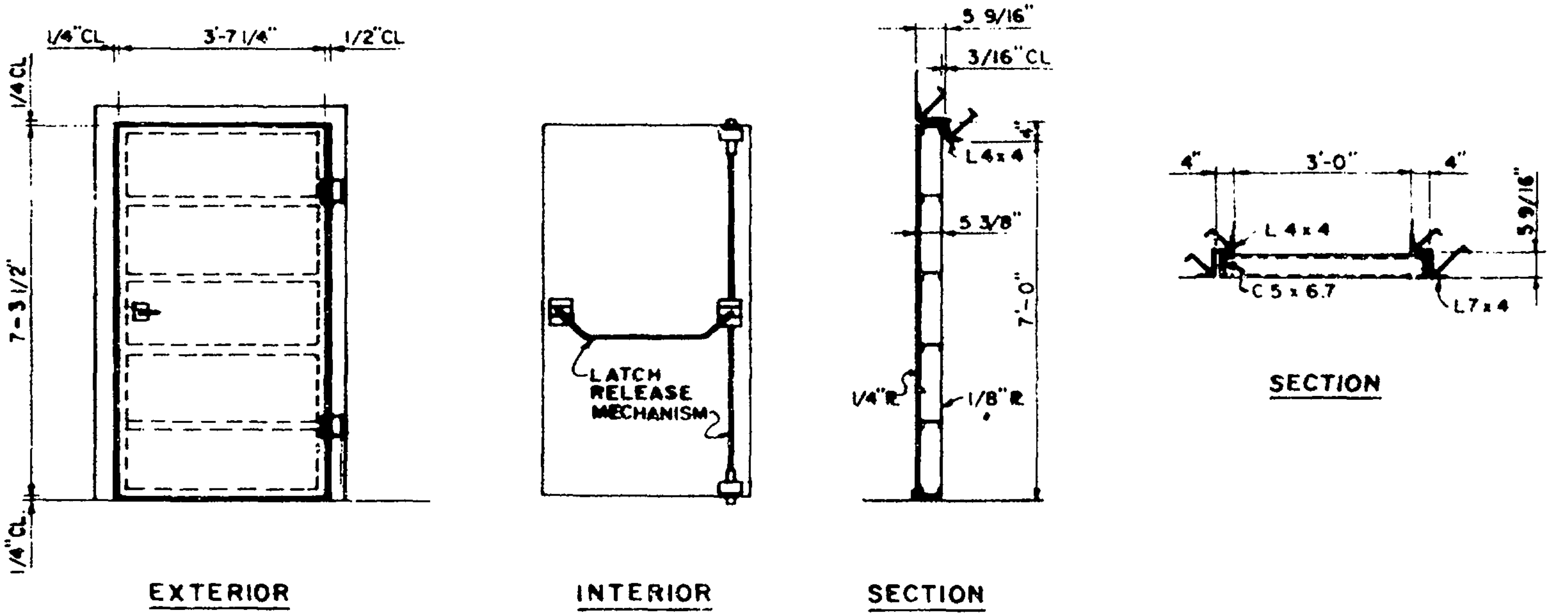

1/4"CL
3'-7 1/4"
1/2"CL
1/4 CL
7-3 1/2"
1/4"CL
EXTERIOR
LATCH RELEASE MECHANISM
INTERIOR
5 9/16"
3/16"CL
L 4 x 4
5 3/8"
7'-0"
1/4"R
1/8"R
SECTION
4"
3'-0"
4"
L 4 x 4
5 9/16"
C 5 x 6.7
L 7 x 4
SECTION

Figure 3.6 Built-up blast door.

## 3.7 Special Provisions for Cold-Formed Steel Panels

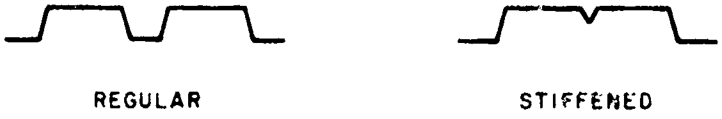
### 3.7.1 General

Cold-formed steel panels are widely used in the construction of industrial installations and pre-engineered buildings as roof and floor decking and wall siding. The behavior of these panels differs significantly from that of hot-rolled structural members due to the cross-sectional shapes, as dictated by the requirements for economical mass production, and to the large width-thickness and depth-thickness ratios of the thin plate elements which make up the cross-sections. For static design, the AISI Specification for the Design of Cold-Formed Steel Members provides the necessary design guidelines. Consideration of the blast-resistant capacity of such panels requires additional provisions as summarized in this section.

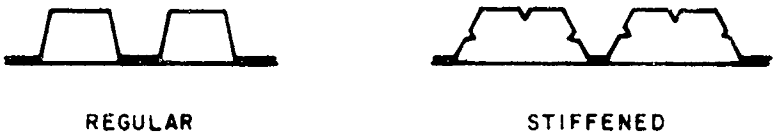
Under static loading, it is known that the load-deflection curve for cold-formed members is markedly non-linear and strongly dependent on the extent of local instability. Effective utilization of the bending properties of cold-formed sections is obtained by accounting for the post-buckling strength of stiffened compressed flanges. This concept, substantiated by numerous tests, is implemented in the AISI Specification.

Cold-formed panels are produced in either open sections forming continuous corrugations, or closed sections consisting of a flat sheet with a series of hat sections. Both types are produced with or without intermediate stiffeners depending on the width-thickness and depth-thickness ratios as illustrated in Fig. 3.7. The amount of deformation that a specific section can develop after reaching its maximum resistance depends on the width-thickness ratio of the flange. For values of $w/t \leq 40$, the behavior is "ductile" and strains several times those attained at the onset of yielding are developed before failure. For larger $w/t$ and especially for values greater than 60, the load-carrying capacity may be reduced abruptly upon yielding of the most stressed outer fiber. For an open cross-section, the descending curve is unstable in nature; whereas for a closed section, the decrease is more gradual, and a certain reserve capacity for energy absorption may exist, enhanced by the catenary action of the flat sheet, as illustrated in Fig. 3.8(b). For this reason, it is recommended that cold-formed steel panels used in blast-resistant structures be chosen from the closed type. Detailed sketches of two typical panel cross-sections are shown in Fig. 3.7(c).

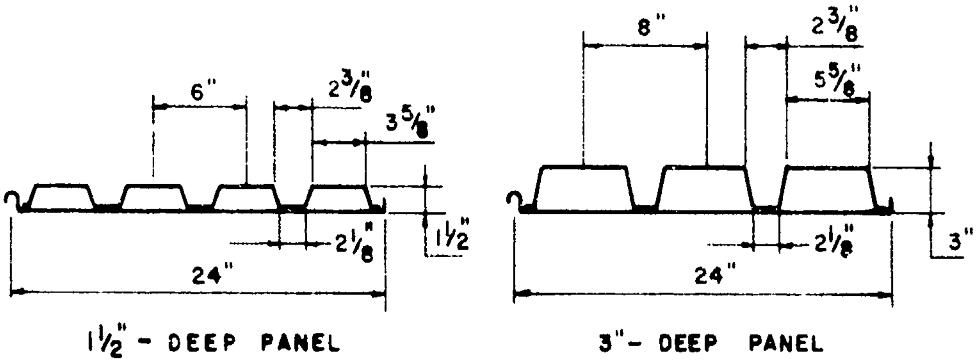
As stated previously, the AISI Specification is based on the recognition and utilization of post-buckling strength. On the


REGULAR
STIFFENED

(a) OPEN HAT SECTIONS


REGULAR
STIFFENED

(b) CLOSED SECTIONS
(HAT SECTIONS & FLAT SHEET)


6"
2 3/8"
3 5/8"
1 1/2"
2 1/8"
24"
1 1/2" - DEEP PANEL
8"
2 3/8"
5 5/8"
3"
2 1/8"
24"
3"- DEEP PANEL

(c) DETAIL OF TYPICAL PANEL
CROSS - SECTIONS

Figure 3.7 Cold-formed sections and typical panel configuration.

other hand, due to the lack of sufficient ductility or adequate rotation capacity, plastic design techniques are not presently included in the Specification.

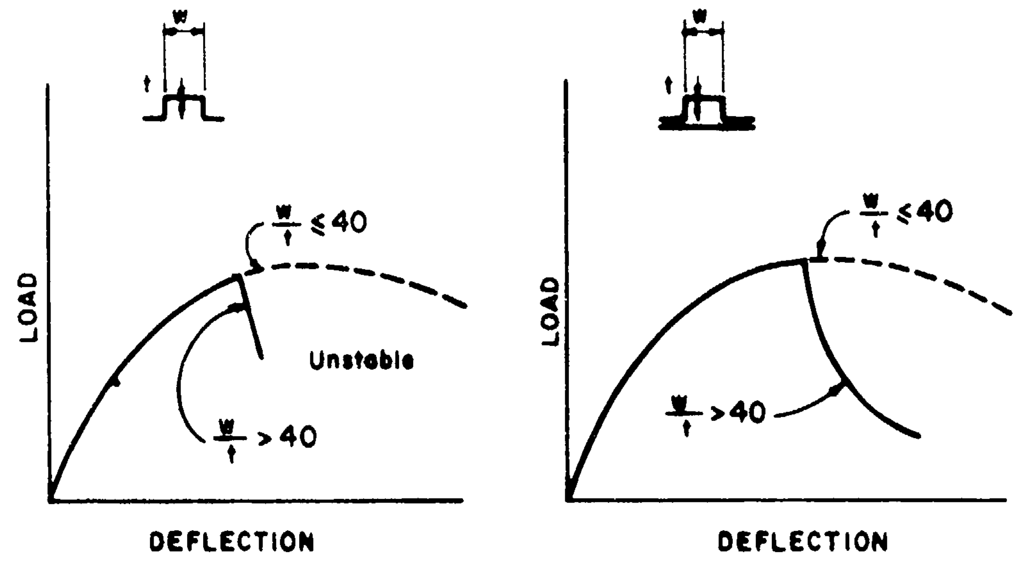
Plastic design is based on the proposition that a flexural element continues to function with large deformations until yielding has practically reached the neutral axis from both sides, thus forming a plastic hinge. In that respect, normal plastic design techniques are not directly applicable to cold-formed construction. This is due to the fact that the width-thickness and/or the depth-thickness ratios are generally greatly in excess of the limits imposed by the requirements for plastic hinge formation and development; and, in most practical cases, local inelastic buckling of flanges or webs occurs first.

Successive formation of plastic hinges in a continuous member or structure produces a redistribution of moments permitting the utilization of the full capacity of more cross-sections of the member at ultimate load and results in a more economical design. Whereas tests have shown that hot-rolled members exhibit sufficient plastic rotation capability and ductility to warrant the successive formation of plastic hinges, this cannot generally be said for cold-formed shapes. It is possible, however, to account for a relatively more limited, but definite amount of plastic behavior in these shapes.

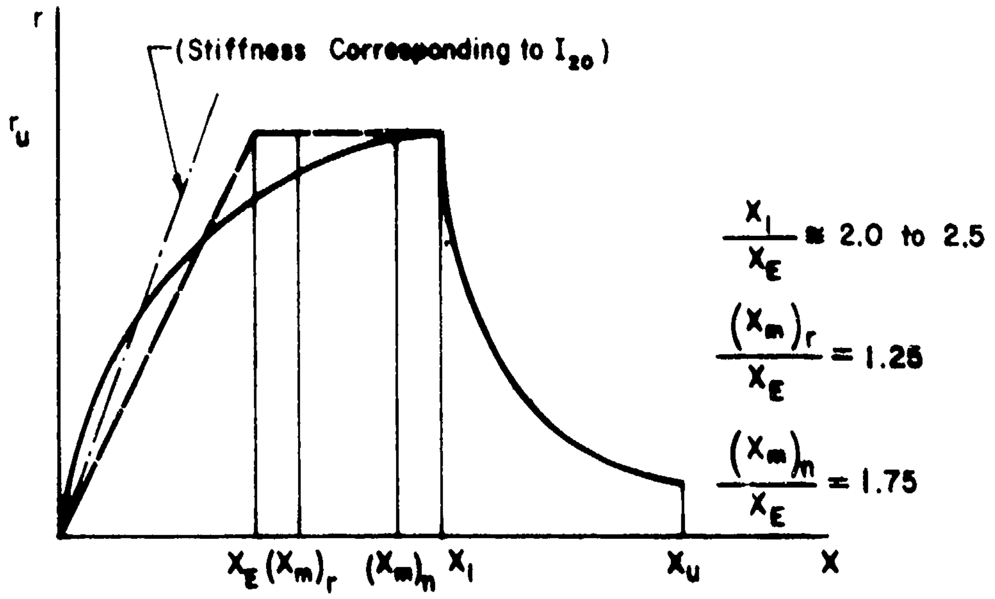
At present, there is limited data on the load-deformation response of cold-formed sections. Consequently, it is impossible to determine the exact nature and extent of moment redistribution in a continuous member of that type. However, the concept of limited ductility and partial redistribution may be adopted when designing panels to resist blast overpressures, allowing for a moderate shift in the elastic moment diagram, reducing the value of maximum bending moments at the supports with appropriate increases for the moments in the spans.

Recent studies have shown that the effective width relationships for cold-formed light-gage elements under dynamic loading do not differ significantly from the static relationships. Consequently, the recommendations presented in the AISI Specification are used as the basis for establishing the special provisions needed for the design of cold-formed panels to resist pressure-time loadings. Some of the formulas of the Specification have been extended to comply with ultimate load conditions, and to permit limited performance in the inelastic range.

Two main modes of failure can be recognized, one governed by flexure and the other by shear. In the case of continuous members, the interaction of the two influences plays a major role



(a) BEHAVIOR OF OPEN SECTIONS (b) BEHAVIOR OF CLOSED SECTIONS



(c) BILINEAR APPROXIMATION OF RESISTANCE-DEFLECTION CURVE FOR CLOSED SECTIONS.

Figure 3.8 Load-deflection characteristics of cold-formed, light-gage sections.

in determining the behavior and the ultimate capacity. Due to the relatively thin webs encountered in cold-formed members, special attention must also be paid to crippling problems. Basically the design will be dictated by the capacity in flexure but subject to the constraints imposed by shear resistance and local stability.

### 3.7.2 Resistance in Flexure

The material properties of the steel used in the production of cold-formed steel panels conform to ASTM Specification A446. This standard covers three grades (a, b and c) depending on the yield point. Most commonly, panels are made of steel complying with the requirements of Grade a, with a minimum yield point of 33 ksi and an elongation of rupture of 20 percent for a 2-inch gage length.

In calculating the dynamic yield stress of cold-formed steel panels, it is recommended that a dynamic increase factor of 1.1 be applied irrespective of actual strain rate, and consequently, the value to be used in design will be

$$F_{dy} = 1.1F_y \tag{3.21}$$

and hence, $F_{dy}$ equals 36.3 ksi for the particular case of a 33-ksi steel.

Ultimate design procedures combined with the effective width concept are used in evaluating the strength of cold-formed light-gage elements. Thus, a characteristic feature of cold-formed elements is the variation of their section properties with the intensity of the load. As the load increases beyond the level corresponding to the occurrence of local buckling, the effective area of the compression flange is reduced; and as a result, the neutral axis moves toward the tension flange with the effective properties of the cross-section such as A, I and S, decreasing with load increase. The properties of panels, as tabulated by the manufacturer, are related to different stress levels: the value of S referring to that of the effective section modulus at ultimate, and the value of I related to a service stress level of 20 ksi. In the case of panels fabricated from hat sections and a flat sheet, two section moduli are tabulated, $S^+$ and $S^-$, referring to the effective section modulus for positive and negative moments, respectively. Consequently, the following ultimate moment capacities are obtained:

$$M_{up} = F_{dy}S^+ \tag{3.22}$$

$$M_{un} = F_{dy}S^- \tag{3.23}$$

where $M_{up}$ = ultimate positive moment capacity for one-foot width of panel

$M_{un}$ = ultimate negative moment capacity for one-foot width of panel

It should be noted that in cases where tabulated section properties are not available, the required properties may be calculated based upon the relationships in the AISI Design Specification.

As for any single-span flexural element, the panel may be subjected to different end conditions, either simply supported or fixed. The fixed-fixed condition is seldom found in practice since this situation is difficult to achieve in actual installations. The simple-fixed condition is found, because of symmetry, in each span of a two-span continuous panel. For multi-span members (three or more), the response is governed by that of the first span, which is generally characterized by a simply-supported condition at one support and a partial moment restraint at the other. Three typical cases can be therefore considered:

1. Simply supported at both ends (single span)
2. Simply supported at one end and fixed at the other (two equal-span continuous member)
3. Simply supported at one end and partially fixed at the other (first span of an equally-spaced multi-span element).

The resistance of the panel is a function of both the strength of the section and the maximum moment in the member. As stated before, moment redistribution is dependent on adequate ductility in plastic hinge regions. Because of the limited ductility of cold-formed elements, the concept suggested here is similar to the one adopted in reinforced concrete design, i.e., partial moment redistribution, wherein the design starts with the elastic moment distribution with further reduction of the maximum negative moments at the supports, and appropriate increases applied to the in-span moments.

A reduction of 15 percent in the negative moment at the inner support of a two equal-span continuous panel is considered acceptable whereas only a reduction of 10 percent in the negative moment of the inner support of the first span of a multi-span (three or more) continuous panel is recommended. Using these recommendations, the computed maximum resistances of a two-span or a multi-span panel become very close to one another, and for simplicity, average values shall be used to cover both cases.

Consequently, for design purposes, the following resistance formulas are recommended:

1. Simply-supported, single-span panel

$$r_u = (8.0\ M_{up})/L^2 \qquad (3.24)$$

2. Simple-fixed, single-span panel or first span of equally-spaced continuous panel

$$r_u = (12.2\ M_{up})/L^2$$

or (3.25)

$$r_u = (10.1\ M_{un})/L^2$$

whichever governs, and where $r_u$ is the resistance per unit length of the panel.

As previously mentioned, the behavior of cold-formed sections in flexure is non-linear as shown in Figs. 3.8(a) and 3.8(b). A bilinear approximation of the resistance-deflection curve is assumed for design. The equivalent elastic deflection $X_E$ is obtained by using the following equation:

$$X_E = (\beta r_u L^4)/EI_{eq} \qquad (3.26)$$

where $\beta$ is a constant depending on the support conditions as follows:

$\beta$ = 0.0130 for simply-supported elements

$\beta$ = 0.0062 for simple-fixed or continuous elements, and

$$I_{eq} = 0.75\ I_{20}$$

where $I_{20}$ is defined as the effective moment of inertia of the section at a service stress of 20 ksi. The value of $I_{20}$ is generally tabulated as a section property of the panel.

Figure 3.8(c) illustrates the non-linear character of the resistance-deflection curve and the suggested bilinear approximation. $X_1$ is defined as the maximum deflection at maximum resistance, and $X_u$ is the ultimate deflection after the drop in load-carrying capacity. Based on experimental evidence, the ratio of $X_1/X_E$ has been estimated to range between 2.0 and 2.5. The amount of plastic deformation which is acceptable in design will vary in magnitude depending on the reusability or non-reusability of the panel after an accidental explosion.

The extent of plastic behavior is expressed in terms of a ductility ratio $\mu = X_m/X_E$. In Fig. 3.8(c), $(X_m)_r$ and $(X_m)_n$ designate the maximum deflections for reusable and non-reusable elements, respectively. According to the criteria in Section 2.3.3, the following design ductility ratios are recommended:

$$\mu = (X_m)_r/X_E = 1.25 \text{ for reusable}$$

and (3.27)

$$\mu = (X_m)_n/X_E = 1.75 \text{ for non-reusable.}$$

These values represent reasonable estimates of the actual capacity of the member to undergo limited plastic deformations. The maximum displacements are kept below the deflection corresponding to maximum resistance in order to prevent any serious impairment to the element. Furthermore, the area beyond $(X_m)_n$ and the descending curve, up to complete loss of carrying capacity is considered a reserve capacity for energy absorption and a safeguard against total collapse.

In addition, in order to restrict the magnitude of rotation at the supports, limitations are placed on the maximum deflections, namely:

$$(X_m)_r = L/130 \text{ or } \theta_{max} = 0.9^\circ$$

(3.28)

$$(X_m)_n = L/65 \text{ or } \theta_{max} = 1.8^\circ$$

for reusable and non-reusable elements, respectively. The values for maximum rotations of cold-formed panels are less than those recommended for hot-rolled sections due to the experimental observation of small deflections at failure for that type of element.

When performing a one-degree-of-freedom analysis of the panel's behavior, the properties of the equivalent system can be evaluated by using a load-mass factor $K_{LM} = 0.74$ which is an average value applicable to all support conditions. The natural period of vibration for the equivalent single-degree system is thus obtained by

$$T_N = 2\pi\sqrt{0.74mL/K_E} \qquad (3.29)$$

where $m = w/g$ is the unit mass of the panel and $K_E = r_uL/X_E$ is the equivalent elastic stiffness of the system.

The problem of rebound should be considered in the design of decking due to the different section properties of the panel, depending on whether the hat section or the flat sheet is in compression. Figure 3.9 presents the maximum elastic resistance in rebound as a function of $T/T_N$. In the practical design range, $T/T_N$ is generally larger than 1.0, and hence, the required elastic resistance in rebound will be less than $0.7r_u$. Most decks have section properties that will provide an elastic resistance in rebound at least equal to this value. However, for the few cases where the actual resistance of the deck in rebound will be smaller than that indicated on the chart of Fig. 3.9, plastic behavior in rebound must be considered and checked against the design criteria. While the behavior in rebound does not often control, the designer should be aware of the problem. In any event, there is a need for providing connectors capable of resisting up-lift or pull-out forces due to load reversal in rebound.

In conclusion, due to the limited amount of experimental data available on the performance of cold-formed, light-gage elements in the inelastic domain, the overall level of confidence in the design of that type of element is considered to be lower than that of hot-rolled sections. Rather than alter the basic design criteria on this count, it is recommended that the peak pressure of the pressure-time loading obtained from TM 5-1300 shall be multiplied by an increase factor of 1.1 for the design of the panels only.

### 3.7.3 Resistance in Shear

In cold-formed steel construction, the designer is faced with somewhat different problems from those encountered in hot-rolled members since webs with h/t values in excess of 60 are common and, at the same time, the fabrication process makes it impractical to use stiffeners. The design web stresses must therefore be limited to insure adequate stability without the aid of stiffeners, thereby preventing premature local web failure and the accompanying loss of load-carrying capacity.

The possibility of web buckling due to bending stresses exists and the critical bending stress is given by

$$F_{cr} = 640{,}000/(h/t)^2 \leq F_y \qquad (3.30)$$

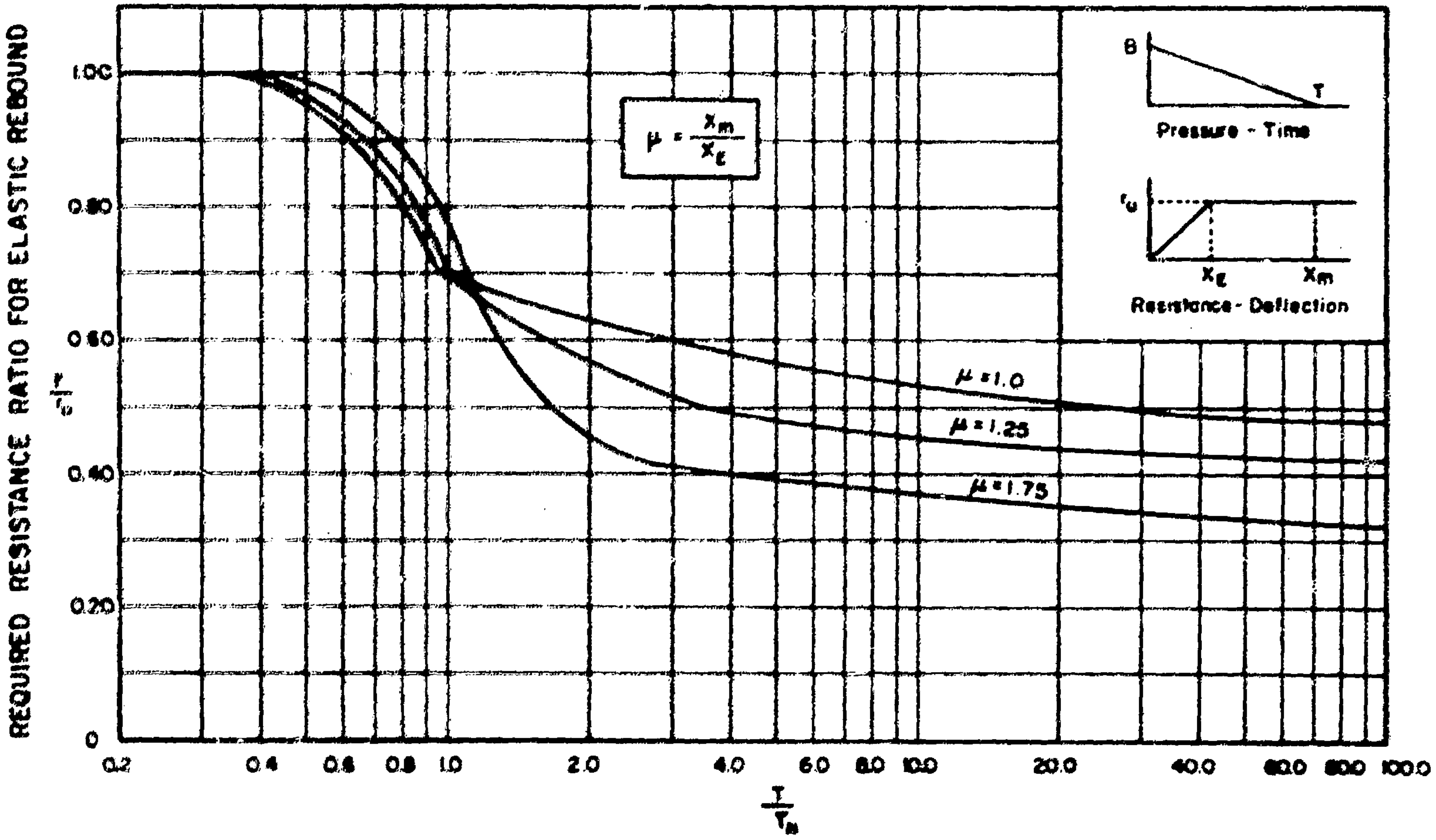


Figure 3.9 Elastic rebound of single-degree-of-freedom system.

Equating $F_{cr}$ to 32 ksi (a stress close to the yielding of the material), a value of h/t = 141 is obtained. Since it is known that webs do not actually fail at these theoretical buckling stresses due to the development of post-buckling strength, it can be safely assumed that webs with h/t $\leq$ 150 will not be susceptible to flexural buckling. Moreover, since the AISI recommendations prescribe a limit of h/t = 150 for unstiffened webs, this type of web instability need not be considered in design.

Panels are generally manufactured in geometrical proportions which preclude web-shear problems when used for recommended spans and minimum support-bearing lengths of 2 to 3 inches. In blast design, however, because of the greater intensity of the loading, the increase in required flexural resistance of the panels calls for shorter spans. As a result, the problem of shear resistance is magnified and requires special treatment.

In most cases, the shear capacity of a web is dictated by instability due to either:

a. Simple shear stresses, or

b. Combined bending and shearing stresses.

For the case of simple shear stresses, as encountered at an end support, it is important to distinguish three ranges of behavior depending on the magnitude of h/t. For large values of h/t, the maximum shear stress is dictated by the elastic buckling in shear; for intermediate h/t values, the inelastic buckling of the webs governs; whereas for very small values of h/t, local buckling will not occur and failure will be caused by yielding produced by shear stresses. The provisions of the AISI Specification in this area are based on a safety factor ranging from 1.44 to 1.67 depending upon h/t. For blast-resistant design, the recommended design stresses for simple shear are based on an extension of the AISI provisions to comply with ultimate load conditions. The specific equations for use in design are summarized in Table 3.4(a).

At the interior supports of continuous panels, high bending moments combine with large shear forces, and webs must be checked for buckling due to combined bending and shear. The interaction formula presented in the AISI Specification is given in terms of allowable stresses rather than critical stresses which produce buckling. In order to adapt this interaction formula to ultimate load conditions, the problem of inelastic buckling under combined stresses has been considered in the development of the recommended design data.

In order to minimize the amount and complexity of design calculations, the allowable dynamic design shear stresses at the interior support of a continuous member have been computed for different depth-thickness ratios and tabulated in Table 3.4(b).

TABLE 3.4

DYNAMIC DESIGN SHEAR STRESSES FOR WEBS
OF COLD-FORMED MEMBERS ($F_y = 33.0$ KSI)

(a) Simple Shear

$$(h/t) \leq 63 \quad F_{dv} = 0.50F_{dy} \leq 18.0 \text{ ksi}$$

$$63 < (h/t) \leq 93 \quad F_{dv} = (190\sqrt{F_{dy}})/(h/t)$$

$$= (1.14 \times 10^3)/(h/t)$$

$$93 < (h/t) \leq 150 \quad F_{dv} = (1.07 \times 10^5)/(h/t)^2$$

(b) Combined Bending and Shear

| (h/t) | $F_{dv}$(ksi) |
|---|---|
| 20 | 9.00 |
| 30 | 8.95 |
| 40 | 8.85 |
| 50 | 8.75 |
| 60 | 8.65 |
| 70 | 8.50 |
| 80 | 8.30 |
| 90 | 8.10 |
| 100 | 7.85 |
| 110 | 7.60 |
| 120 | 7.30 |

In addition to shear problems, concentrated loads or reactions at panel supports, applied over relatively short lengths, can produce load intensities that can cripple unstiffened thin webs. As stated in the commentary of the AISI Specification, a theoretical analysis of the phenomenon of web crippling is extremely complex since it involves elastic and inelastic instability under non-uniform stress distribution, combined with local yielding in the immediate region of load application. The AISI recommendations have been developed by relating extensive experimental data to service loads with a safety factor of 2.2 which was established taking into account the scatter in the data. For blast design of cold-formed panels, it is recommended that the AISI values be multiplied by a factor of 1.50 in order to relate the crippling loads to ultimate conditions with sufficient provision for scatter in test data.

Since it has been recommended that closed section panels (flat sheet and hat sections) be used, only those equations pertaining to webs restrained from rotation will be considered. Using a yield stress of 33.0 ksi and the factor of 1.50 mentioned above, the ultimate crippling loads are given as follows:

a. Acceptable ultimate end support reaction

$$Q_u = 49.5t^2(4.44 + 0.558\sqrt{N/t}) \qquad (3.31)$$

b. Acceptable ultimate interior support reaction

$$Q_u = 49.5t^2(6.66 + 1.446\sqrt{N/t}) \qquad (3.32)$$

where $Q_u$ = ultimate support reaction

$N$ = bearing length

$t$ = web thickness

The charts in Figs. 3.10(a) and 3.10(b) present the variation of $Q_u$ as a function of the web thickness for bearing lengths from 1 to 5 inches, for end and interior supports, respectively. It should be noted that the values reported in the charts relate to one web only, the total ultimate reaction being obtained by multiplying $Q_u$ by the number of webs in the panel.

In design, the maximum shear forces and dynamic reactions are computed as a function of the maximum resistance in flexure. The ultimate load-carrying capacity of the webs of the panel must then be compared with these forces. As a general comment, the shear capacity is controlled by simple shear buckling or web crippling for simply-supported elements and by the allowable design shear stresses at the interior supports for continuous panels.

In addition, it can be shown that the resistance in shear governs only in cases of relatively very short spans. If a design is controlled by shear resistance, it is recommended that another panel be selected since a flexural failure mode is generally preferred. However, for existing installations that are to be checked for their structural strength in a certain pressure range, the maximum resistance of the panel may be determined by either flexure or shear, whichever controls.

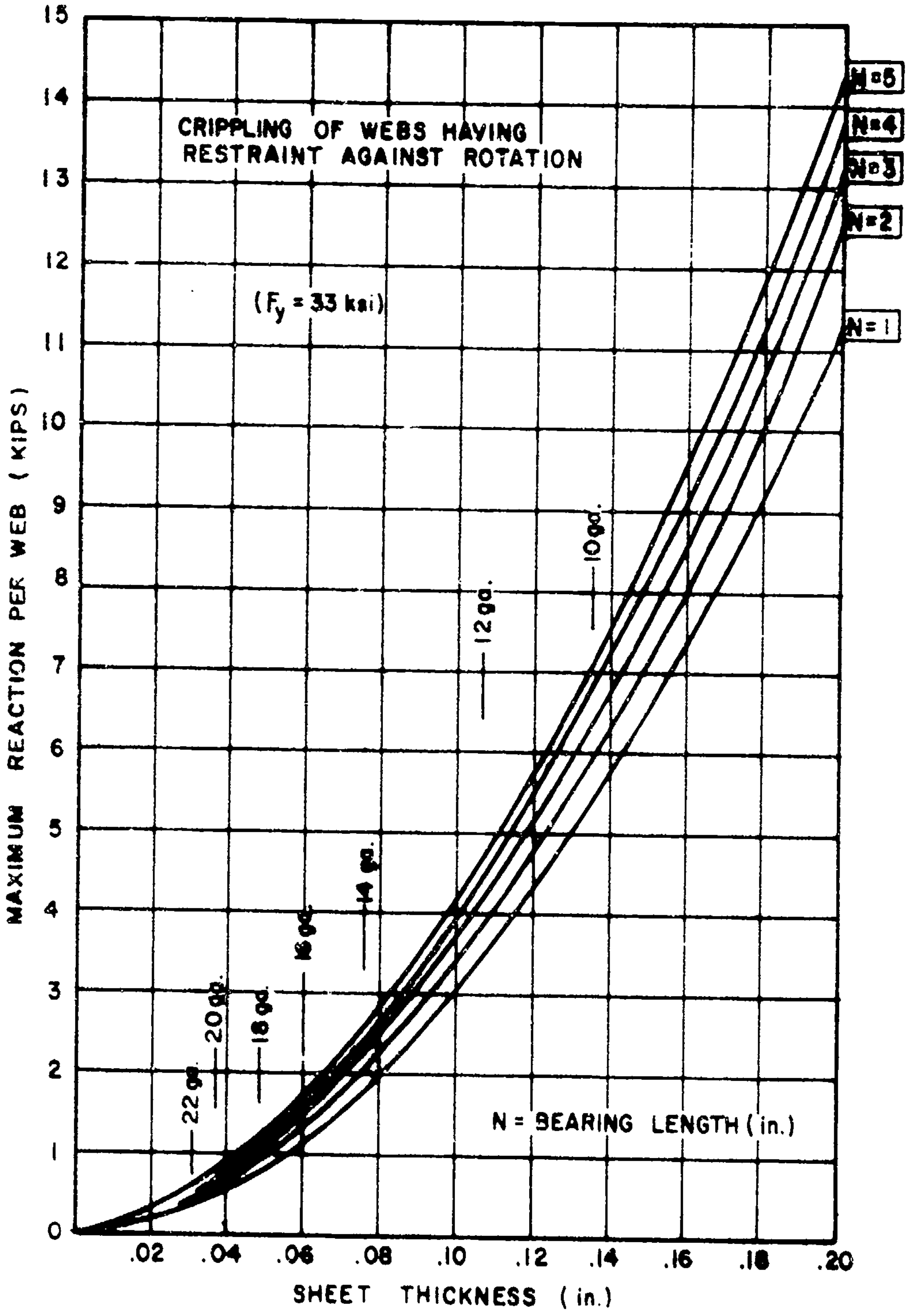


Figure 3.10(a) Maximum end support reaction for cold-formed steel sections.

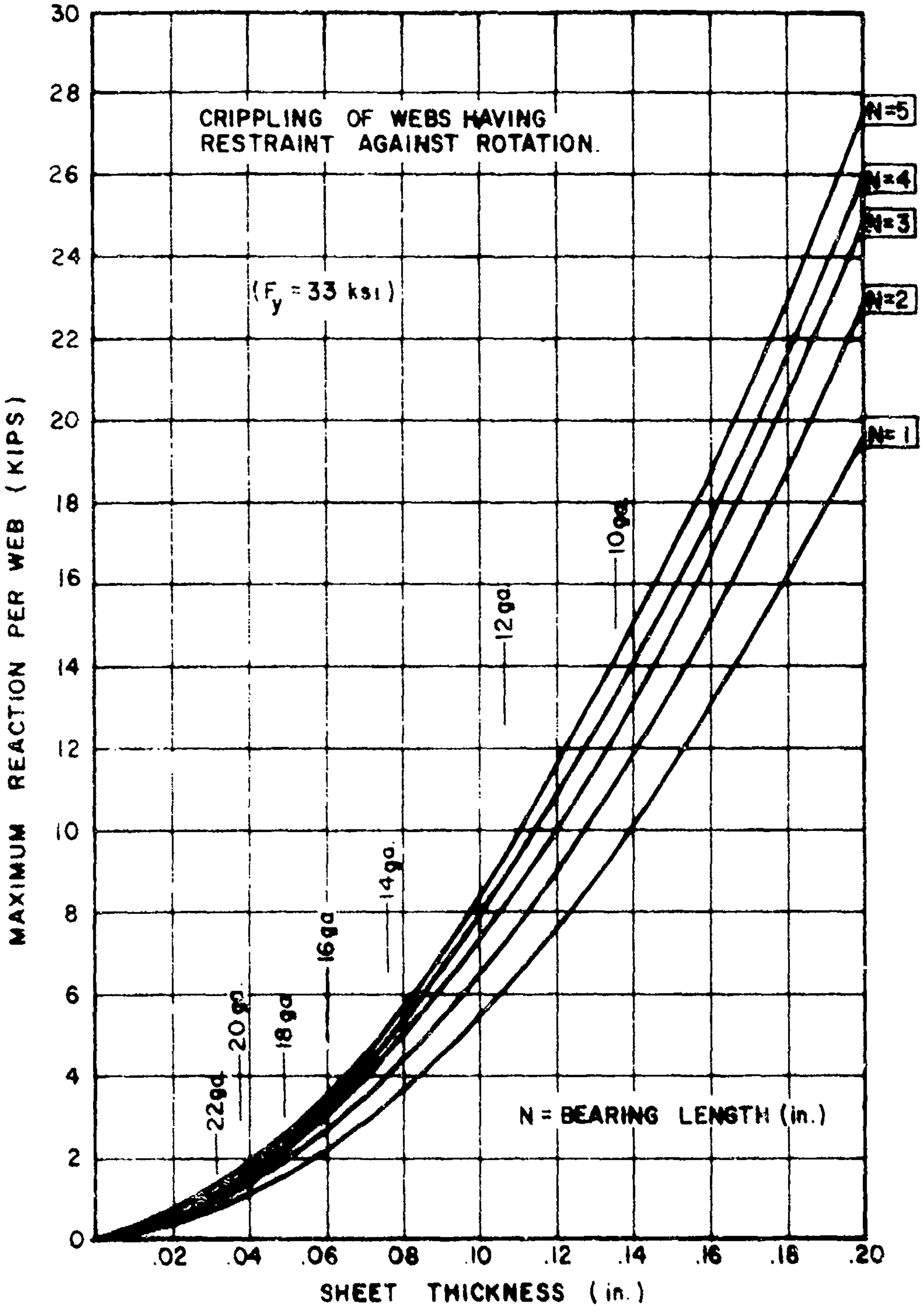


Figure 3.10(b) Maximum interior support reaction for cold-formed steel sections.

## 3.8 Special Provisions for Open-Web Steel Joists

Open-web joists are commonly used as load-carrying members for the direct support of roof and floor deck in buildings. The design of joists for conventional loads is covered by the "Standard Specification for Open Web Steel Joists, J-Series and H-Series", adopted by the Steel Joist Institute and the American Institute of Steel Construction. For blast design, all the provisions of this Specification are in force, except as modified herein.

These joists are manufactured using either hot-rolled or cold-formed steel. H-series joists are composed of 50-ksi steel in the chord members and either 36-ksi or 50-ksi steel for the web sections. J-series joists have 36-ksi steel throughout.

Standard load tables are available for simply-supported, uniformly-loaded joists supporting a deck and so constructed that the top chord is braced against lateral buckling. These tables indicate that the capacity of a particular joist may be governed by either flexural or shear (maximum end reaction) considerations. As discussed previously, it is preferable in blast applications to select a member whose capacity is controlled by flexure.

The tabulated loads include a check on the bottom chord as an axially-loaded tensile member and the design of the top chord as a column or beam column. The width-thickness ratios of the unstiffened or stiffened elements of the cross-section are also limited to values specified in the Standard Specification for Joists.

The dynamic ultimate capacity of open-web joists may be taken equal to 1.87 times the load given in the joist tables. This value of 1.87 represents the safety factor of 1.7 multiplied by a dynamic increase factor of 1.1.

The adequacy of the section in rebound must be evaluated. Upon calculating the required resistance in rebound, $\bar{r}/r_u$, using the rebound chart in Chapter 6 of TM 5-1300 (Fig. 6-8), the lower chord must be checked as a column or beam column. If the bottom chord of a standard joist is not adequate in rebound, the chord must be strengthened either by reducing the unbraced length or by increasing the chord area. The top chord must be checked as an axial tensile member but in most circumstances, it will be adequate.

The bridging members required by the joist specification should be checked for both the initial and rebound phase of the response to verify that they satisfy the required spacing of compression flange bracing for lateral buckling.

The joist tables indicate that the design of some joists is governed by failure of the web bar members in tension or compression near the supports. In such cases, the ductility ratio for the joist should not exceed unity. In addition, the joist members near the support should be investigated for the worst combination of slenderness ratio and axial load under load reversal.

For hot-rolled members not limited by shear considerations, design ductility ratios up to the values specified in Chapter 2 can be used. The design ductility ratio of joists with light gage chord members should be limited to 1.0.

The top and bottom chords should be symmetrical about a vertical axis. If double angles or bars are used as chord members, the components of each chord should be fastened together so as to act as a single member.

## CHAPTER 4

## COLUMNS AND BEAM-COLUMNS

### 4.1 Introduction

This chapter considers the dynamic design of steel columns and beam-columns. Except as modified herein, the provisions on plastic design of the current issue of the AISC Specification apply. In addition to the provisions set forth in this chapter, the members should satisfy the requirements of Section 3.3.4, Local Buckling, and Section 3.3.6, Lateral Bracing.

The buckling strength of columns subjected to dynamic loading is dependent not only upon the magnitude of the pressure pulse but also upon the rise time of the loading and the duration of the loading pulse relative to the natural frequency of the member. For a short duration load, a column will sustain a greater load than under static conditions, since inertial forces retard the buckling process, thereby providing a stabilizing effect. Hence, in blast design, a reasonable approach will be to base the calculations upon the static formulas for steel columns presented in Part 2 of the AISC Specification with the modification that the static yield stress is replaced by the comparable dynamic value.

### 4.2 Plastic Design Criteria

The design criteria for columns and beam-columns must account for their behavior not only as individual members but also as members of the overall frame structure. Within a rigid frame, the individual members are subjected to various combinations of axial load, bending moments and end restraint. In addition, their stability is affected by lateral torsional buckling and the overall frame characteristics in terms of sway stability and lateral deflection.

Depending upon the nature of the loading, several design cases may be encountered. Summarized below are the necessary equations for the dynamic design of steel columns and beam-columns.

(1) In the plane of bending of compression members which would develop a plastic hinge at ultimate loading, the slenderness ratio $l/r$ shall not exceed $C_c$ defined by:

$$C_c = \sqrt{2\pi^2 E/F_{dy}} \qquad (4.1)$$

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where E is the modulus of elasticity of steel in ksi (29,000) and $F_{dy}$ is the dynamic yield stress (see Chapter 2).

(2) The ultimate strength of an axially loaded compression member should be taken as:

$$P_u = 1.7\ AF_a \tag{4.2}$$

where A is the gross area of the member and $F_a$ is given by

$$F_a = \frac{\left[1 - \dfrac{(Kl/r)^2}{2C_c^2}\right]F_{dy}}{\dfrac{5}{3} + \dfrac{3(Kl/r)}{8\ C_c} - \dfrac{(Kl/r)^3}{8C_c^3}} \tag{4.3}$$

where $Kl/r$ is the largest effective slenderness ratio. See Section 4.3.

(3) Members subject to combined axial load and biaxial bending moment should be proportioned so as to satisfy the following set of interaction formulas, i.e.; Equation 4.4 in all cases and either Equation 4.5(a) or Equation 4.5(b), whichever applies.

$$\frac{P}{P_u} + \frac{C_{mx}M_x}{(1 - P/P_{ex})M_{mx}} + \frac{C_{my}M_y}{(1 - P/P_{ey})M_{my}} \leq 1 \tag{4.4}$$

$$\frac{P}{P_p} + \frac{M_x}{1.18M_{px}} + \frac{M_y}{1.18M_{py}} \leq 1.0 \quad \text{for } P/P_p \geq 0.15 \qquad [4.5(a)]$$

or

$$\frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \leq 1 \quad \text{for } P/P_p < 0.15 \qquad [4.5(b)]$$

where $M_x$, $M_y$ = maximum applied moments about the x and y axes

$P$ = applied axial load

$P_{ex} = 23/12\ AF'_{ex}$; $P_{ey} = 23/12\ AF'_{ey}$

$F'_{ex} = 12\pi^2E/[23(Kl_b/r_x)^2]$

$F'_{ey} = 12\pi^2 E/[23(Kl_b/r_y)^2]$

$l_b$ = actual unbraced length in the plane of bending

$r_x$, $r_y$ = corresponding radii of gyration

$P_p = F_{dy}A$

$C_{mx}$, $C_{my}$ = coefficients applied to bending term in interaction formula and dependent upon column curvature caused by applied moments (see Section 1.6.1 of the AISC Specification).

$M_{px}$, $M_{py}$ = plastic bending capacities about the x and y axes = $F_{dy}Z_x$, $F_{dy}Z_y$, respectively

$M_{mx}$, $M_{my}$ = moments that can be resisted by the member in the absence of axial load

For columns braced in the weak direction

$$M_{mx} = M_{px};\ M_{my} = M_{py} \qquad (4.6)$$

For columns unbraced in the weak direction

$$\begin{aligned} M_{mx} &= [1.07 - (l/r_y)\sqrt{F_{dy}}/3160]M_{px} \leq M_{px} \\ M_{my} &= [1.07 - (l/r_x)\sqrt{F_{dy}}/3160]M_{py} \leq M_{py} \end{aligned} \qquad (4.7)$$

Subscripts x and y indicate the axis of bending about which a particular design property applies.

Columns may be considered braced in the weak direction when the provisions of Section 3.3.6 are satisfied.

Beam-columns should also satisfy the requirements of Section 3.3.3, Design for Shear.

## 4.3 Effective Length Ratios for Beam-Columns

The basis for determining the effective lengths of beam-columns for use in the calculation of $P_u$, $P_{ex}$, $P_{ey}$, $M_{mx}$ and $M_{my}$ in plastic design is outlined below.

For plastically designed braced and unbraced planar frames which are supported against displacement normal to their plane, the effective length ratios in Tables 4.1 and 4.2 shall apply. Table 4.1 corresponds to bending about the strong axis of a member, while Table 4.2 corresponds to bending about the weak axis. In each case, $l$, is the distance between points of lateral support corresponding to $r_x$ or $r_y$, as applicable. The effective length factor, K, in the plane of bending shall be governed by the provisions of Section 4.4.

TABLE 4.1

EFFECTIVE LENGTH RATIOS FOR BEAM-COLUMNS (WEBS OF MEMBERS IN THE PLANE OF THE FRAME, i.e., BENDING ABOUT THE STRONG AXIS)

| | Braced Planar Frames* | One- and Two-Story Unbraced Planar Frames* |
|---|---|---|
| $P_u$ | Use larger ratio, $l/r_y$ or $l/r_x$ | Use larger ratio, $l/r_y$ or $Kl/r_x$ |
| $P_{ex}$ | Use $l/r_x$ | Use $Kl/r_x$ |
| $M_{my}$ | Use $l/r_y$ | Use $l/r_y$ |

* $l/r_x$ shall not exceed $C_c$.

TABLE 4.2

EFFECTIVE LENGTH RATIOS FOR BEAM-COLUMNS (FLANGES OF MEMBERS IN THE PLANE OF THE FRAME, i.e., BENDING ABOUT THE WEAK AXIS)

| | Braced Planar Frames* | One- and Two-Story Unbraced Planar Frames* |
|---|---|---|
| $P_u$ | Use larger ratio, $l/r_y$ or $l/r_x$ | Use larger ratio, $l/r_x$ or $Kl/r_y$ |
| $P_{ey}$ | Use $l/r_y$ | Use $Kl/r_y$ |
| $M_{my}$ | Use $l/r_x$ | Use $l/r_x$ |

* $l/r_y$ shall not exceed $C_c$.

For columns subjected to biaxial bending, the effective lengths given in Tables 4.1 and 4.2 apply for bending about the respective axes, except that $P_u$ for unbraced frames shall be based on the larger of the ratios $Kl/r_x$ or $Kl/r_y$. In addition, the larger of the slenderness ratios $l/r_x$ or $l/r_y$ shall not exceed $C_c$.

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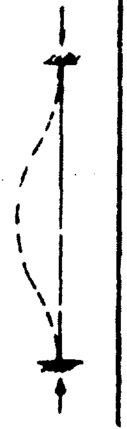
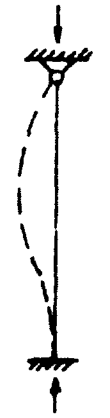
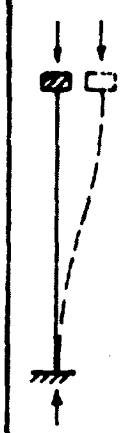
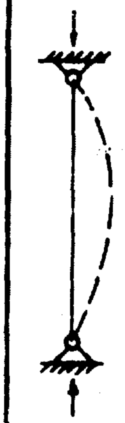
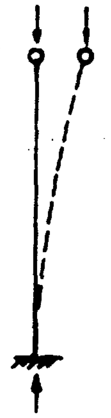
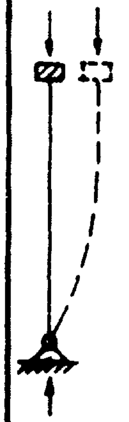
## 4.4 Effective Length Factor, K

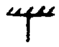
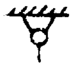
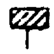
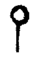
In braced frames wherein lateral stability is provided by adequate attachment to diagonal bracing, shear walls, an adjacent structure having adequate lateral stability, floor slabs or roof decks secured horizontally by walls, or bracing systems parallel to the plane of the frame, the effective length factor, K, for the compression members should be taken as unity, unless analysis shows that a smaller value may be used.

However, when a column or a beam-column is a member of an unbraced frame, the K factor can be greater than unity depending upon the degree and nature of fixity at the ends of the member. There are various methods for evaluating the effective length factor including some relatively recent developments which are gaining recognition, such as the inelastic K-factor concept and the story buckling strength concept. The best source of authoritative data in the area of K factors is the Commentary to the AISC Specification, with Supplements and other AISC literature.

In plastic design, it is usually sufficiently accurate to use the K factors from Table C 1.8.1 of the AISC Manual for the condition closest to that in question rather than to refer to the alignment chart (Figure C 1.8.2 of the AISC Manual). It is permissible to interpolate between different conditions in Table 4.3 (Table C 1.8.1 of the AISC Manual) using engineering judgment. In general, a design K value of 1.5 is conservative for the columns of unbraced frames when the base of the columns is assumed pinned, since conventional column base details will usually provide partial rotational restraint at the column base. For the girders of unbraced frames, a design K value of 0.75 is recommended.

### TABLE [illegible].3 EFFECTIVE LENGTH FACTORS FOR COLUMNS AND BEAM-COLUMNS.

| | (a) | (b) | (c) | (d) | (e) | (f) |
|---|---|---|---|---|---|---|
| Buckled shape of column is shown by dashed line |  |  |  |  |  |  |
| Theoretical K value. | 0.5 | 0.7 | 1.0 | 1.0 | 2.0 | 2.0 |
| Recommended design value when ideal conditions are approximated. | 0.65 | 0.80 | 1.2 | 1.0 | 2.10 | 2.0 |

| | | |
|---|---|---|
| End condition code. |  | Rotation fixed and translation fixed. |
| |  | Rotation free and translation fixed. |
| |  | Rotation fixed and translation free. |
| |  | Rotation free and translation free. |

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# CHAPTER 5

# FRAME DESIGN

## 5.1 Introduction

The dynamic plastic design of frames for blast-resistant structures is treated in this chapter. The material is oriented toward industrial building applications common to ammunition manufacturing and storage facilities, i.e., relatively low, predominantly single-story, multi-bay structures. This treatment applies principally to acceptor structures subjected to incident overpressures at barricaded distances, or face-on overpressures at unbarricaded distances, i.e. about 10 psi.

The design of blast-resistant frames is characterized by the following:

a. Simultaneous application of vertical and horizontal pressure-time loadings with peak overpressures considerably in excess of conventional loads.

b. Design criteria permitting inelastic local and overall dynamic structural deformations (deflections and rotations).

c. Design requirements dictated by the operational needs of the facility and, often, the need for reusability, with minor repair work, following an accidental explosion.

A wide variety of solutions exist for the overall framing system chosen for a particular application. This choice is governed by the strength, stiffness and stability requirements for the structure in response to the imposed blast loading as defined by the peak overpressure, the load duration and the orientation of the path of the blast wave with respect to the axes of the structure. Other equally important influences include operational and architectural requirements, the desired post-accident condition and economy of material and fabrication.

Rigid frame construction is recommended in the design of blast-resistant structures since this system provides open interior space combined with substantial resistance to lateral forces. In addition, this type of construction possesses inherent energy absorption capability due to the successive development of plastic hinges up to the ultimate capacity of the structure.

Where the interior space and wall opening requirements permit, it may be effective to provide bracing. Since one- and two-story frames do not normally require bracing to prevent frame instability under gravity and combined gravity and lateral loads, this bracing can consist of relatively slender tension diagonals designed to supplement the stiffness of the rigid frame. Lateral deflections and joint moments are thereby decreased, permitting smaller member sizes. This concept of supplementary bracing often proves economical in frames wherein the columns are subjected to bending about their weak axes. In addition, for low pressures, there may be cost advantages in also providing fixed beam to column connections throughout the braced frame. In such cases, the major part of the horizontal stiffness is furnished by the bracing.

The particular objective in the remainder of this chapter is to provide rational procedures for efficiently performing the preliminary design of blast-resistant frames. Rigid frames are considered in Section 5.2 and frames with supplementary bracing and rigid or non-rigid connections are treated in Section 5.3. In both cases, preliminary dynamic load factors are provided for establishing equivalent static loads for both the local and overall frame mechanisms. Based upon the mechanism method, as employed in static plastic design, rational estimates for the required plastic bending capacities are developed. Approximate values are established for the axial forces and shears in the frame members. The dynamic deflections and rotations in the sidesway and local beam mechanism modes are estimated, based upon single degree-of-freedom analyses. Throughout this development the design criteria of Chapter 2 and the procedures for the design of individual members in Chapters 3 and 4 apply.

In order to confirm that a trial frame design meets the recommended deformation criteria (Section 2.3) and to verify the adequacy of the member sizes established on the basis of estimated dynamic forces and moments, a rigorous frame analysis should be run with a dynamic analysis program such as DYNFA. This program, developed for Picatinny Arsenal by Ammann & Whitney, performs the multi-degree-of freedom, nonlinear, dynamic analysis of braced and unbraced, rigid and non-rigid frames of one or more stories.

## 5.2 Preliminary Design of Single-Story Rigid Frames

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### 5.2.1 Collapse Mechanisms for Rectangular Multi-Bay Frames

General expressions for the possible collapse mechanisms of single-story rigid frames are presented in Table 5.1 for pinned and fixed base frames subjected to combined vertical and horizontal loading. The governing failure mode in a particular application depends upon the overall frame geometry, the relation between member properties within the frame and the ratio of the peak horizontal to vertical blast load. The design objective is then to proportion the frame members such that the governing mechanism represents an economical solution.

For a particular frame within a framing system, the ratio of total horizontal to vertical peak loading, denoted by $\alpha$, is influenced by the horizontal framing plan of the structure and is determined as follows:

$$\alpha = q_h/q_v \qquad (5.1)$$

where $q_v = p_v b_v$ = peak vertical load on rigid frame

$q_h = p_h b_h$ = peak horizontal load on rigid frame

$p_v$ = blast over-pressure on roof

$p_h$ = reflected blast pressure on front wall

$b_v$ = tributary width for vertical loading

$b_h$ = tributary width for horizontal loading

The $\alpha$ ratio will usually lie in the range from about 1.8 to 2.5 when the blast wave is directed perpendicular to the roof purlins. The value of $\alpha$ is much higher when the direction of the blast is parallel to the purlins, since in this case only part of the vertical load is carried by the girders of the rigid frame in the direction of the load.

It is assumed that the plastic bending capacity of the roof girder, $M_p$, is constant for all bays. The capacity of the exterior and interior columns are taken as $CM_p$ and $C_1M_p$, respectively. Since the exterior column is generally subjected to reflected pressures, it is recommended that a value of C greater than 1.0 be selected, i.e. the bending capacity of the exterior column should be greater than that of the roof girder. Therefore, for the purpose of analysis, the plastic hinge at the blastward haunch is assumed to form in the girder, although

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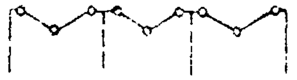
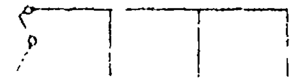
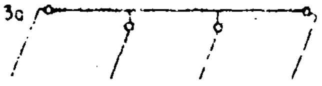
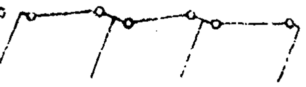
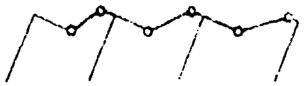
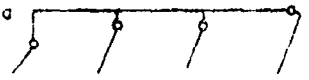
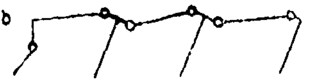
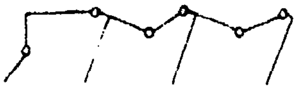
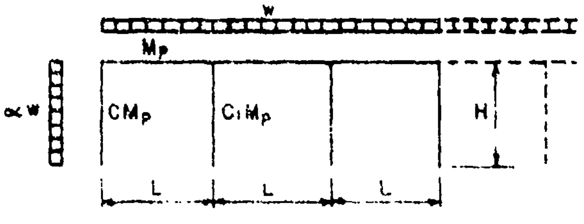
TABLE 5.1 COLLAPSE MECHANISMS FOR RIGID FRAMES WITH PINNED BASES.

| Collapse Mechanism | Plastic Moment $M_p$ | |
|---|---|---|
| | Pinned Bases | Fixed Bases |
| 1  BEAM MECHANISM | $\frac{wL^2}{16}$ | $\frac{wL^2}{16}$ |
| 2  BEAM MECHANISM | $\frac{\alpha w H^2}{4(2C+1)}$ | $\frac{\alpha w H^2}{4(3C+1)}$ |
| 3a  PANEL MECHANISM | $\frac{\alpha w H^2}{2} \cdot \frac{1}{2+(n-1)C_1}$ <br> $(C_1 \gtrless 2)$* | $\frac{\alpha w H^2}{4} \cdot \frac{1}{1+(n-1)C_1+C}$ <br> $(C_1 \gtrless 2)$* |
| 3b  PANEL MECHANISM | $\frac{\alpha w H^2}{4n}$ <br> $(C_1 \geq 2)$* | $\frac{\alpha w H^2}{2} \cdot \frac{1}{2(n+C)+(n-1)C_1}$ <br> $(C_1 \geq 2)$* |
| 4  COMBINED MECHANISM | $\frac{w}{8n}(\alpha H^2 + \frac{n}{2}L^2)$ | $\frac{w}{2} \cdot \frac{\alpha H^2 + \frac{n}{2}L^2}{2(2n+C)+(n-1)C_1}$ |
| 5a  COMBINED MECHANISM | $\frac{\frac{3}{8}\alpha w H^2}{C+\frac{1}{2}+\frac{C_1}{2}(n-1)}$ <br> $(C_1 \gtrless 2)$* | $\frac{\frac{3}{8}\alpha w H^2}{\frac{5}{2}C+(n-1)C_1+\frac{1}{2}}$ <br> $(C_1 \gtrless 2)$* |
| 5b  COMBINED MECHANISM | $\frac{\frac{3}{8}\alpha w H^2}{C+(n-\frac{1}{2})}$ <br> $(C_1 \geq 2)$* | $\frac{\frac{3}{8}\alpha w H^2}{\frac{5}{2}C+(n-1)\frac{C_1}{2}+(n-\frac{1}{2})}$ <br> $(C_1 \geq 2)$* |
| 6  COMBINED MECHANISM | $\frac{\frac{w}{8}[3\alpha H^2+(n-1)L^2]}{C+(2n-\frac{3}{2})}$ | $\frac{\frac{w}{8}[3\alpha H^2+(n-1)L^2]}{\frac{5}{2}C+(n-1)\frac{C_1}{2}+(2n-\frac{3}{2})}$ |



n = Number of bays = 1,2,3...

w = Uniform equivalent static load

* For $C_1 = 2$ hinges form in the girders and columns at interior joints

this assumption may not result in an economical design if the span of the exterior bay is substantially greater than the height of the frame.

The resistance for each mechanism may be computed by equating the work done by the external loads to the internal work absorbed by each plastic hinge as it rotates. For example, for the roof girder fixed-ended beam mechanism of a single-bay frame, the external work, $W_E$, is given by $(wL^2\theta)/4$ where $\theta$ is the rotation of a member in the assumed collapse mechanism. The internal work, $W_I$, is given by $4M_p\theta$. By equating the external work to the internal work, the moment capacity corresponding to this mechanism is $wL^2/16$.

As seen from Table 5.1, the mechanism that corresponds to the lowest possible resistance is a function of the relative magnitudes of the horizontal and vertical forces, the height and length of a bay, the number of bays, and the ratios, C and $C_1$, of the plastic bending capacities of the columns to the plastic bending capacities of the girders. It is evident that the choice of C and $C_1$ determines whether certain hinges will form in the girder or in the column or in both simultaneously. However, for analytical purposes, the hinge may be assumed to be concentrated at the haunch. The values of C and $C_1$ have a major influence on the weight and behavior of the frame since they establish the relative bending capacities for the girders and the columns.

It will normally be uneconomical to proportion a frame subjected to vertical and horizontal blast pressures so that the mode of failure is a simple beam mechanism since this implies that either the roof girder or the columns will be designed to remain elastic. It will generally be found that a combined sidesway mechanism, with hinges in both the girders and the columns provides greater overall economy.

In analyzing a given frame with certain member properties, the controlling mechanism is the one with the lowest resistance. In design, however, the load is fixed and the required design plastic moment is the largest $M_p$ value obtained from all possible mechanisms. For that purpose C and $C_1$ should be selected so as to minimize the value of the maximum required $M_p$ from among all the possible mechanisms. Reasonable values of C and $C_1$ can be established by substituting assumed values of these constants in the expressions given in Table 5.1 and calculating $M_p$. After a few trials, it will become obvious which choice of C and $C_1$ tends to minimize the largest value of $M_p$. Rigorous minimum weight design procedures could be used to establish C and $C_1$, but such procedures are relatively complex and time consuming and are not warranted for a preliminary design.

### 5.2.2 Dynamic Deflections and Rotations

Static design is based primarily on strength considerations with some servicibility limitations on the deflections and rotations at the service load level and consequently, the deformations associated with the collapse mechanism are not usually calculated. However, in the dynamic design of steel frames which are permitted to deflect well into the plastic range, it is essential to compute the maximum inelastic deflections and rotations since these quantities, in addition to the strength requirements, constitute the basic design parameters.

As discussed previously, it will normally be more economical to proportion the members so that the controlling failure mechanism is a combined mechanism rather than a beam mechanism. The mechanism having the least resistance constitutes an acceptable mode of failure provided that the magnitudes of the maximum deflections and rotations do not exceed the maximum values recommended in Chapter 2 of this report.

### 5.2.3 Dynamic Load Factors

For the purpose of preliminary design, it is necessary to make certain initial assumptions regarding the dynamic effect of the load on the deflection of the frame. These assumptions are required since the natural period of the system is initially unknown. To obtain initial estimates of the required mechanism resistance, the dynamic load factors of Table 5.2 may be used to obtain equivalent static loads for the indicated mechanisms. These factors were obtained from single-degree-of-freedom analyses of several steel frames in an intermediate to low pressure range. For example, considering the sidesway behavior of a single-story, multi-bay frame with pinned bases, the parameter $T/T_N$ is usually about 0.4 where $T$ is the load duration and $T_N$ is the natural period of vibration for the frame. For the ductility ratio of 3 corresponding to a reusable structure, Figure 6-7 of TM 5-1300 indicates that $B/R_u = 2$ which corresponds to a dynamic load factor ($R_u/B$) of 0.5.

The preliminary load factors in Table 5.2 are necessarily approximate and make no distinction for different end conditions. However, they are expected to result in reasonable estimates of the required resistance for a trial design. Once the trial member sizes are established, then the natural period and deflection of the frame can be calculated using the procedure and data of Section 5.2.5.

TABLE 5.2

DYNAMIC LOAD FACTORS (DLF) AND EQUIVALENT
STATIC LOADS FOR TRIAL DESIGN

| COLLAPSE MECHANISM | STRUCTURE REUSABLE | STRUCTURE NON-REUSABLE |
|---|---|---|
| Beam | 1.0 | 0.8 |
| Panel or Combined | 0.5 | 0.35 |

Equivalent static vertical load = $q_v$ x DLF = w

Equivalent static horizontal load = $q_h$ x DLF = $\alpha$w

The dynamic load factors of Table 5.2 are presented for both reusable and non-reusable frames. In each case, the factors for a panel or combined sidesway mechanism are lower than those for a beam mechanism since the natural period of a sidesway mode will normally be greater than the natural periods of the individual elements.

### 5.2.4 Preliminary Sizing of Frame Members

To obtain preliminary estimates of the peak axial forces in the girders, the approximate formulas of Figure 5.1 may be used. Figure 5.1 may also be used for calculating preliminary values of the peak shears in the columns. In applying Figure 5.1 in preliminary design, the equivalent horizontal static load shall be computed using the dynamic load factor for a panel or combined sidesway mechanism.

Preliminary values of the peak axial loads in the columns and peak shears in the girders may be computed by multiplying

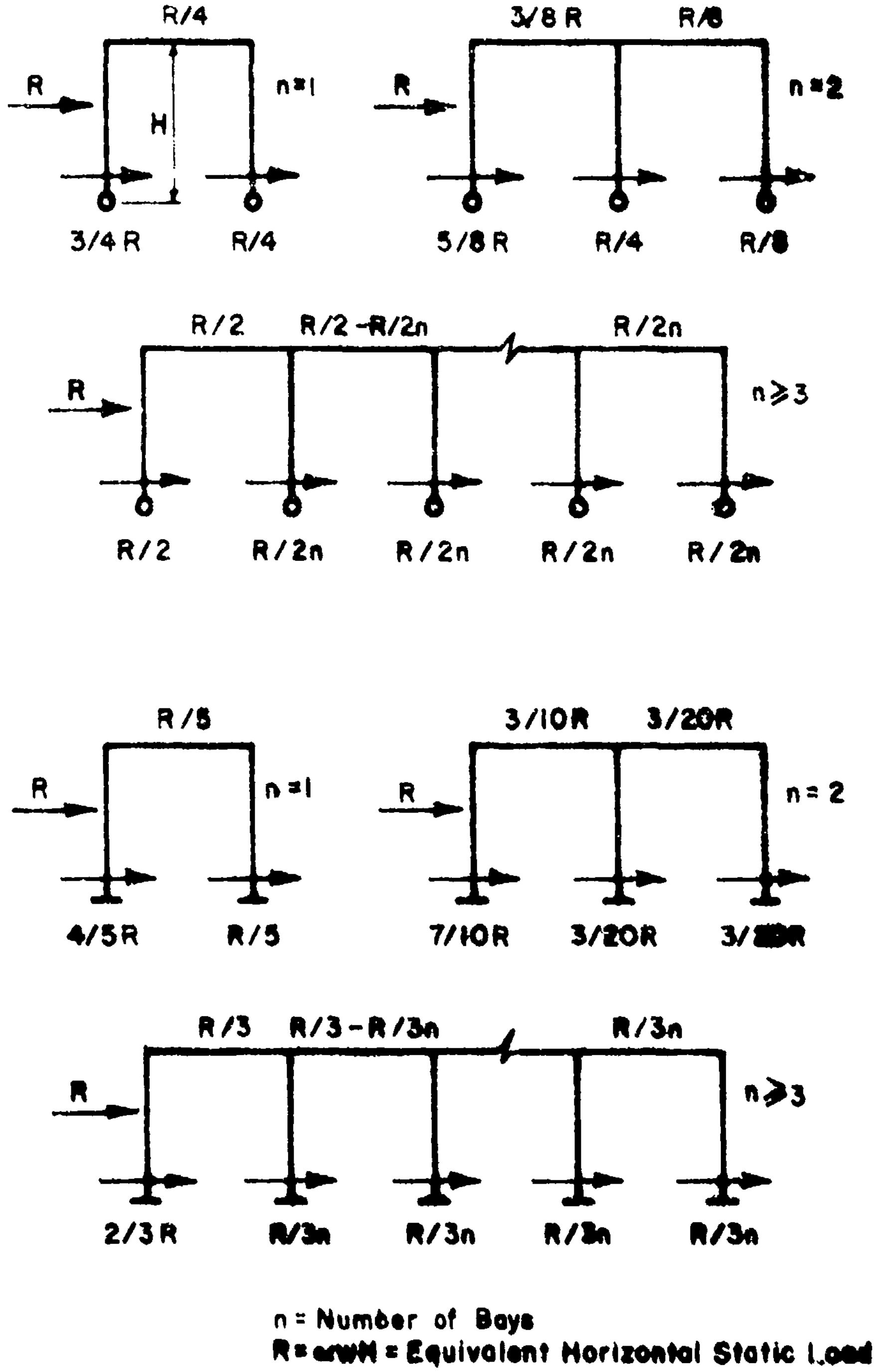

R/4
R
n=1
H
3/4 R
R/4
3/8 R
R/8
R
n=2
5/8 R
R/4
R/8
R/2
R/2 - R/2n
R/2n
R
n≥3
R/2
R/2n
R/2n
R/2n
R/2n
R/5
R
n=1
4/5R
R/5
3/10R
3/20R
R
n= 2
7/10R
3/20R
3/20R
R/3
R/3 - R/3n
R/3n
R
n≥3
2/3 R
R/3n
R/3n
R/3n
R/3n
n = Number of Bays
R = αWH = Equivalent Horizontal Static Load

Figure 5.1 Preliminary values of peak shears and axial loads due to horizontal loading of single-bay and multi-bay frames.

the equivalent vertical static load by the roof tributary area. Since the axial loads in the columns are due to the reaction from the roof girders, the equivalent static vertical load should be computed using the dynamic load factor for a beam mechanism.

Each member in a frame under the action of horizontal and vertical blast loads is subjected to combined bending moments and axial loads. Therefore, the resistance of a frame depends upon the ultimate strength of the members acting as beam-columns.

Due to the dynamic nature of the blast loads, the phasing between critical values of the axial force and bending moment cannot be established from a simplified analysis. Therefore, it is recommended that the axial loads and moments computed in accordance with the above recommendations be assumed to act concurrently for the purpose of beam-column design. This assumption tends to be conservative.

When a blast wave impinges on the building from a quartering direction, the columns and the girders in the exterior frames are subject to biaxial bending due to the simultaneous action of vertical pressures on the roof and horizontal pressures on the exterior walls. In such cases, the moments and forces are calculated by analyzing the response of the frame in each direction for the appropriate components of the load. The results in each direction are then superimposed in order to perform the analysis or design of the beams, columns and beam-columns of the structure. This approach is generally conservative since it assumes that the peak values of the forces in each direction occur simultaneously throughout the three-dimensional structure.

Having estimated the maximum values of the forces and moments throughout the frame, the preliminary sizing of the members can be performed using the criteria in Chapter 2 and the procedures for beams, columns and beam-columns in Chapters 3 and 4.

### 5.2.5 Stiffness and Deflection

Since the preliminary design of the frame is based on assumed dynamic load factors, it is desirable to make an approximate calculation of the sidesway deflection of the frame prior to performing a rigorous analysis on a computer. The deflection of a possible local mechanism of the roof girder or front column

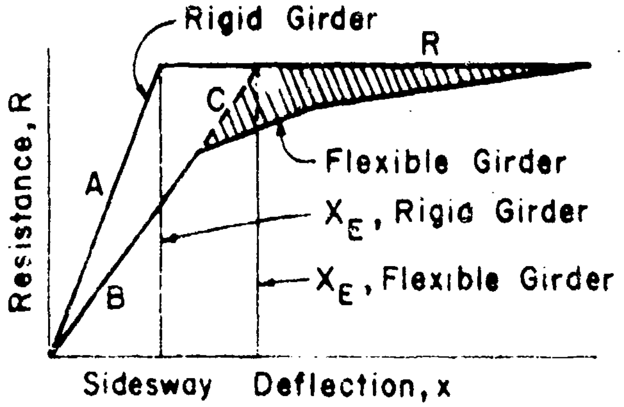


Figure 5.2 Effect of girder flexibility on resistance-deflection function of multi-bay frames in sidesway.

should also be computed. The latter can be carried out using the appropriate stiffness and resistance functions for these elements in conjunction with Chapter 3 of this report and with the dynamic response charts in Chapter 6 of TM 5-1300.

To estimate the sideway deflection of the frame, the flexibility of the girders should be taken into account; otherwise, the computed displacements may be underestimated considerably. An approximate method which accounts for girder flexibility is presented below. Figure 5.2 presents the form of the resistance-deflection diagram for a typical multi-column frame subjected to lateral load. Line A represents the resistance for infinite girder stiffness. With infinite girder stiffness, plastic hinges would develop simultaneously at both ends of all columns. If the actual girder flexibility is considered, the hinges would be found to develop successively as indicated by Line B. The recommended resistance diagram is Line C, an extension of the initial slope of Line B to the intersection with the line of maximum resistance. The shaded area represents the error introduced. Use of Line C will result in a calculated deflection which is smaller than the true deflection; however, the error involved is generally small. To obtain the effective spring constant K considering girder flexibility, it is necessary to determine the slope of Line C. This can be determined by imposing a uniformly distributed horizontal load upon the frame and calculating the corresponding horizontal deflection of the girder.

The stiffness factor K, for single-story rectangular frames subjected to uniform horizontal loading is defined in Table 5.3. Considering an equivalent single-degree-of-freedom system, the sidesway natural period of this frame is

$$T_N = 2\pi \sqrt{m_e/KK_L} \tag{5.2}$$

where $K_L$ is a load factor that modifies K, the frame stiffness due to a uniform load, so that the product, $KK_L$ is the equivalent stiffness due to a unit load applied at the equivalent lumped mass, $m_e$. The load factor is given by

$$K_L = 0.55(1 - 0.25\beta) \tag{5.3}$$

where $\beta$ = base fixity factor (i.e., $\beta = 0$ for a pinned base and $\beta = 1$ for a fixed base). The equivalent mass $m_e$ to be used in calculating the period of a sidesway mode consist of the total roof mass plus one-third of the column and wall masses. Since all of these masses are considered to be concentrated at the roof level the mass factor $K_M$ is equal to 1.0.

The limiting resistance $R_u$ is given by

$$R_u = \alpha wH \tag{5.4}$$

where w is the equivalent static load based on the dynamic load factor for a panel or combined sidesway mechanism.

It should be pointed out that $R_u$ corresponds to the smallest value of the resistance for all possible panel or combined mechanisms as determined from Section 5.2.

The equivalent elastic deflection $X_E$ corresponding to $R_u$ is

$$X_E = R_u/K \tag{5.5}$$

The ductility ratio for the sidesway deflection of the frame can be computed using the dynamic response chart (Figure 6.7) of TM 5-1300.

The maximum deflection $X_m$ is then calculated from

$$X_m = \mu X_E \tag{5.6}$$

where $\mu$ is the ductility ratio in sidesway.

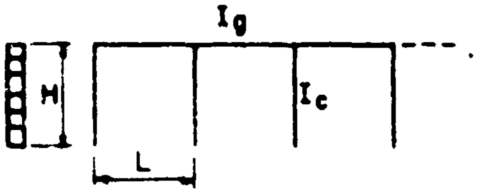
TABLE 5.3 STIFFNESS FACTORS FOR SINGLE-STORY, MULTI-BAY FRAMES SUBJECTED TO UNIFORM HORIZONTAL LOADING.

STIFFNESS FACTOR $K = \frac{E I_{ca}}{H^3} \cdot C_2 \cdot \left[1 + (0.7 - 0.1\beta)(n-1)\right]$

$n$ = Number of Bays

$\beta$ = Base Fixity Factor†

$$D = \frac{I_g / L}{I_{ca}(0.75 + 0.25\beta)/H}$$



$I_{ca}$ = Average Column Moment of Inertia
$= \sum I_c / (n+1)$

| D | $C_2$ | | |
|---|---|---|---|
| | $\beta$ = 1.0 | $\beta$ = 0.5* | $\beta$ = 0 |
| 0.25 | 26.7 | 14.9 | 3.06 |
| 0.50 | 32.0 | 17.8 | 4.65 |
| 1.00 | 37.3 | 20.6 | 6.04 |

* Values of $C_2$ are Approximate for this $\beta$

† $\beta$ = 1.0 For Fixed Base
= 0.0 For Hinged Base

## 5.3 Preliminary Design of Single-Story Frames with Supplementary Bracing

### 5.3.1 Introduction

This section presents an approximate method for the preliminary design of single-story frames with supplementary bracing and either rigid or non-rigid girder to column connections. The procedure is similar to that presented in Section 5.2 for rigid frames except that the formulas are modified to include the effect of bracing. For design purposes, the behavior of these frames can be treated as the combination of two separate load-carrying systems, i.e., rigid frame action plus bracing. Only frames with diagonal X bracing are considered and only the tension diagonal is assumed effective in resisting horizontal load and contributing to the frame stiffness.

In the remainder of this section, it should be noted that whenever the term "braced frame" is used, reference is still being made to this special framing case of frame action with supplementary diagonal bracing.

### 5.3.2 Bracing Ductility Ratio

For these applications, the brace members are not expected to remain elastic under the blast loading. It is therefore necessary to determine if this yielding will be excessive when the system is permitted to deflect to the limits given in Section 2.3.3. On the basis of the analysis summarized below, it can be concluded that the maximum bracing ductility ratio will be acceptable provided that ductile steel bracing members are used.

The ductility ratio associated with tension yielding of the bracing may be expressed as the ratio of the maximum strain in the brace to the yield strain as follows:

$$\mu = \varepsilon_{max}/\varepsilon_y \qquad (5.7)$$

where $\varepsilon_{max}$ is the maximum strain and $\varepsilon_y$ is the yield strain given by $F_{dy}/E$ where $F_{dy}$ is the dynamic yield stress and $E$ is Young's modulus.

Assuming small deflections and neglecting axial deformations in the girders and columns, the maximum strain in a brace is given by

$$\varepsilon_{max} = \delta(\cos^2\gamma)/L \qquad (5.8)$$

where $\delta$ is the sidesway deflection, L is the bay width and $\gamma$ is the vertical angle between the brace and a horizontal plane. The derivation of this relationship is shown in Figure 5.3.

The ductility ratio for the brace may thus be written

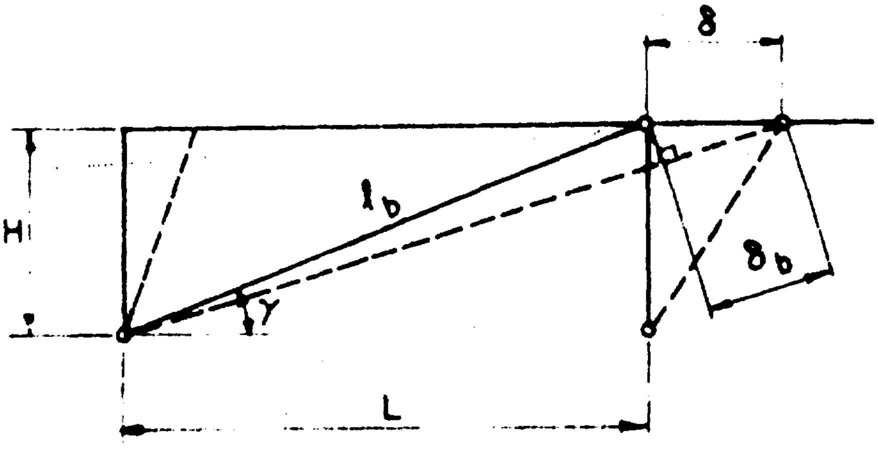
$$\mu = \frac{\delta(\cos^2\gamma)E}{L\,F_{dy}} \qquad (5.9)$$

From the deflection criteria of Chapter 2, $\delta$ is limited to H/50 for reusable structures and to H/25 for non-reusable structures. Substitution of these limiting deflections in the above equation gives

$$\mu = \frac{H}{50L}(\cos^2\gamma)\frac{E}{F_{dv}} \qquad (5.10)$$

for reusable structures, and

$$\mu = \frac{H}{25L}(\cos^2\gamma)\frac{E}{F_{dy}} \qquad (5.11)$$



$$\delta_b = \delta\cos\gamma$$

$$\epsilon_{max.} = \delta_b/\ell_b$$

$$\ell_b = L/\cos\gamma$$

$$\epsilon_{max.} = \delta\cos^2\gamma/L$$

Figure 5.3 Strain in tension brace due to sidesway.

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for non-reusable structures. Taking as an example $F_{dy} = 39.6$ ksi which corresponds to the specified minimum yield stress of A36 steel with a ten percent increase for dynamic effects, and $E = 29{,}000$ ksi, the bracing ductility ratio for a non-reusable structure at the maximum permissible deflection is given by,

$$\mu = 29.2 \frac{H}{L} (\cos^2\gamma) \qquad [5.11(a)]$$

It can easily be verified that the term $\frac{H}{L} (\cos^2\gamma)$ will not exceed about 0.5 for structures with reasonable proportions. Therefore, the bracing ductility ratio will not exceed 14.6 for a non-reusable structure at the maximum permissible deflection. The ductility ratio corresponding to fracture of mild steel in a standard tension test is on the order of 100 to 200. Hence, the bracing ductility ratio at the maximum permissible sidesway deflection is sufficiently low to preclude the possibility of fracture even after several applications of the blast loading.

Equations 5.10 and 5.11 indicate that the bracing ductility ratio is inversely proportional to the dynamic yield stress for the material, i.e., the higher the yield stress, the lower the ductility ratio for a given sidesway deflection. However, high strength steel members are generally not recommended for use as diagonal braces since these steels are not as ductile as mild steel. Similarly, wire rope and structural strand with limited ductility are also not recommended for this application.

### 5.3.3 Collapse Mechanisms for Frames with Supplementary Bracing

The possible collapse mechanisms of single-story frames with diagonal tension bracing are given in Tables 5.4 and 5.5 for pinned-base frames with rigid and non-rigid girder to column connections. In each case, the ultimate capacity is expressed in terms of the equivalent static load and the member ultimate strength (either $M_p$ or $A_bF_{dy}$). In these tables, the cross-sectional area of the tension brace is denoted by $A_b$, the parameter m is the number of braced bays and $F_{dy}$ is the dynamic yield stress for the bracing member. In developing these expressions, the same assumptions were made as in Section 5.2.1, i.e., $M_p$ for the roof girder is constant for all bays, the bay width, L, is constant and the column moment capacity

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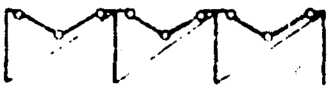
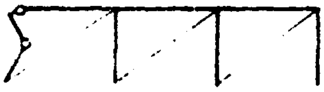
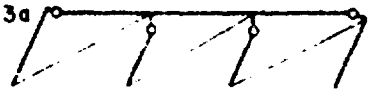
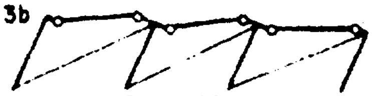
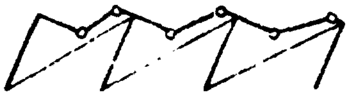
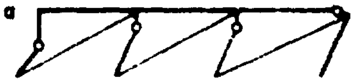
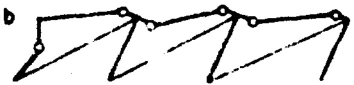
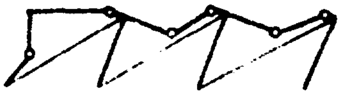
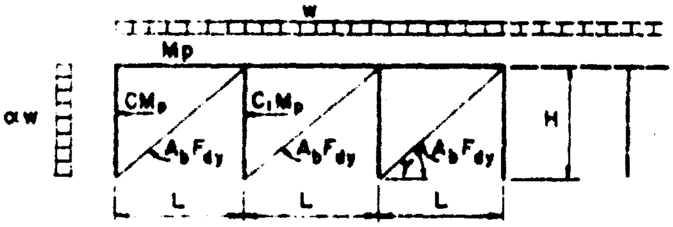
## TABLE 5.4 COLLAPSE MECHANISMS FOR RIGID FRAMES WITH SUPPLEMENTARY BRACING AND PINNED BASES.

| Collapse Mechanism | Plastic Moment $M_p$ |
|---|---|
| 1  BEAM MECHANISM | $\frac{wL^2}{16}$ |
| 2  BEAM MECHANISM | $\frac{\alpha w H^2}{4(2C+1)}$ |
| 3a  PANEL MECHANISM | $\left(\frac{\alpha w H^2}{2} - m A_b F_{dy} H \cos\gamma\right) \cdot \frac{1}{2+(n-1)C_1}$ <br> $(C_1 \leq 2)^*$ |
| 3b  PANEL MECHANISM | $\frac{\alpha w H^2}{4n} - \frac{m A_b F_{dy} H \cos\gamma}{2n}$ <br> $(C_1 \geq 2)^*$ |
| 4  COMBINED MECHANISM | $\frac{w}{8n}(\alpha H^2 + \frac{n}{2} L^2) - \frac{m A_b F_{dy} H \cos\gamma}{4n}$ |
| 5a  COMBINED MECHANISM | $\frac{\frac{3}{8}\alpha w H^2 - \frac{m}{2} A_b F_{dy} H \cos\gamma}{C + \frac{1}{2} + \frac{C_1}{2}(n-1)}$ <br> $(C_1 \leq 2)^*$ |
| 5b  COMBINED MECHANISM | $\frac{\frac{3}{8}\alpha w H^2 - \frac{m}{2} A_b F_{dy} H \cos\gamma}{C + (n-\frac{1}{2})}$ <br> $(C_1 \geq 2)^*$ |
| 6  COMBINED MECHANISM | $\frac{\frac{w}{8}\left[3\alpha H^2 + (n-1)L\right] - \frac{m}{2} A_b F_{dy} H \cos\gamma}{C + (2n - \frac{3}{2})}$ |

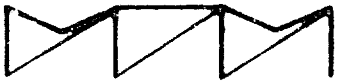
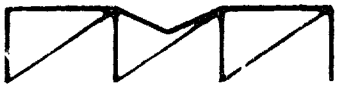
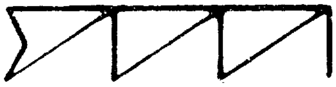
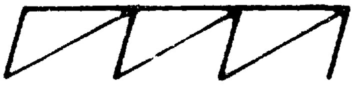
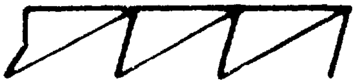


n = Number of bays = 1, 2, 3...

w = Uniform equivalent static load

* For $C_1 = 2$ hinges form in the girders and columns at interior joints

TABLE 5.5 COLLAPSE MECHANISMS FOR FRAMES WITH SUPPLEMENTARY BRACING, NON-RIGID GIRDER TO COLUMN CONNECTIONS AND PINNED BASES.

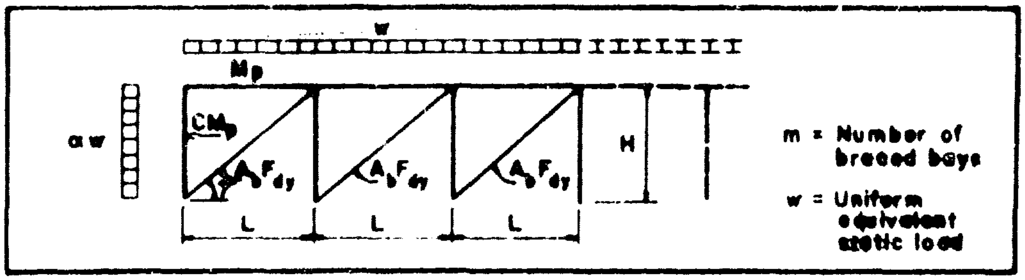
| Collapse Mechanism | Ultimate Capacity | Framing Type |
|---|---|---|
| 1  BEAM MECHANISM EXTERIOR GIRDER | $M_p = wL^2/8$<br>$M_p = wL^2/12$<br>$M_p = wL^2/16$ | ①<br>②<br>③ |
| 2  BEAM MECHANISM INTERIOR GIRDER | $M_p = wL^2/8$<br>$M_p = wL^2/16$ | ①<br>② & ③ |
| 3  BEAM MECHANISM BLASTWARD COLUMN | $M_p = \alpha w H^2/8$<br>$M_p = \frac{\alpha w H^2}{4(2C+1)}$ | ① & ②<br>③ |
| 4  PANEL MECHANISM | $A_b F_{dy} = \alpha w H/2m\cos\gamma$<br>$A_b F_{dy} = \frac{\alpha w H}{2m\cos\gamma} - \frac{2M_p}{mH\cos\gamma}$ | ① & ②<br>③ |
| 5  COMBINED MECHANISM | $A_b F_{dy} = \frac{3\alpha w H}{4m\cos\gamma} - \frac{(2C+1)M_p}{mH\cos\gamma}$ | ③ |

GIRDER FRAMING TYPE:

① GIRDER SIMPLY SUPPORTED BETWEEN COLUMNS

② GIRDER CONTINUOUS OVER COLUMNS

③ GIRDER CONTINUOUS OVER COLUMNS AND RIGIDLY CONNECTED TO EXTERIOR COLUMNS ONLY


w
Mp
αw
CMp
$A_b F_{dy}$
$A_b F_{dy}$
$A_b F_{dy}$
H
L
L
L
m = Number of braced bays
w = Uniform equivalent static load

coefficient, C, is greater than 1.0.

For non-rigid girder to column connections, the resistance functions for local mechanisms of the roof girder depend upon whether the girder is continuous over the columns or is framed between the columns. When the girder is continuous, the interior bay girder mechanisms are the same as that for a fixed beam, while the resistance of the exterior girder is the same as that of a fixed-pinned beam. In certain cases, it may be economical to provide a rigid connection at the exterior haunches and non-rigid connections at interior column lines.

As for the case of a rigid frame, the expressions in Tables 5.4 and 5.5 were computed by equating the work done by the external loads to the internal work absorbed during plastic deformation. For a braced frame, the internal work consists of the energy absorbed by each plastic hinge as it rotates plus the energy absorbed by the tension diagonals as they yield in tension. For a braced frame with non-rigid connections, only the yielding of the tension diagonals contributes to the internal work during plastic deformation of a panel (sidesway) mechanism.

As an example, for the panel mechanism of Table 5.4 with $C_1$ [illegible] 2 (i.e. case 3b) the external work is given by

$$W_E = [illegible] \frac{wH^2\theta}{2}$$

where $\theta$ is the rotation of the columns in the panel mechanism. The internal work performed by the plastic hinges as they rotate through the angle $\theta$ is

$$W_I = 2nM_p\theta$$

where n is the number of bays.

The internal work performed by the tension bracing is given by the simple expression,

$$W_I = m A_b F_{dy} \delta_b$$

where m is the number of braced bays, $F_{dy}$ is the dynamic yield stress for the bracing and $\delta_b$ is the plastic axial elongation. For small deflections

$$\delta_b = H\theta\cos\gamma$$

By equating the external and internal work

$$M_p = \frac{\alpha wH^2}{4n} - \frac{m}{2n} A_b F_{dy} H\cos\gamma$$

For a panel mechanism, with non-rigid girder to column connections (case 4 of Table 5.5), the following resistance function is obtain by equating the work done by the external loads to the energy absorbed by yielding of the tension bracing:

$$A_b F_{dy} = \frac{\alpha wH}{2m\cos\gamma}$$

For rigid frames with tension bracing, it is necessary to vary C, $C_1$ and $A_b$ in order to achieve an economical design. When non-rigid girder to column connections are used, C and $C_1$ drop out of the resistance function for the sidesway mechanism and the area of the bracing can be calculated directly.

### 5.3.4 Dynamic Load Factors

For purposes of a trial design, the dynamic load factors of Table 5.2 may also be used for braced frames. In general, the sidesway stiffness of braced frames will be greater than for unbraced frames and the corresponding panel or sidesway dynamic load factor may also be greater. However, while Table 5.2 is necessarily approximate and serves only as a rational starting point for a preliminary design, refinements to this Table for frames with supplementary diagonal braces are not warranted.

### 5.3.5 Stiffness and Deflection

The provisions of Section 5.2.5, Stiffness and Deflection, may be used for braced frames with the following modifications:

(1) The sidesway natural period of a frame with supplementary diagonal bracing is given by

$$T_N = 2\pi \sqrt{m_e/(KK_L + K_b)} \qquad (5.13)$$

in which $K_b$ is the horizontal stiffness of the tension bracing given by

$$K_b = \frac{nA_b E\cos^3\gamma}{L}$$

and where K, $K_L$ and $m_e$ are the equivalent frame stiffness, frame load factor and effective mass, respectively, as defined in Section 5.2.5.

(2) The elastic deflection of a braced frame is given by

$$X_E = R_u/(KK_L+K_b) \qquad (5.14)$$

Note that the frame stiffness, K, is zero for braced frames with non-rigid connections.

### 5.3.6 Preliminary Sizing of Frame Members

For braced frames, Figure 5.4 may be used to determine values of girder axial loads and column shears for preliminary design. Note from this figure that the shear in the blastward column and the axial load in the exterior girder are the same as for the rigid frame as shown in Figure 5.1. For interior bays the corresponding quantities are reduced by the horizontal component of the force in the brace given by

$$F_H = A_b F_{dy} \sin\theta \qquad (5.15)$$

If a bay is not braced then this quantity should be set equal to zero for the next bay. To avoid an error, horizontal equilibrium should be checked using the formula

$$R = V_1 + nV_2 + mA_b F_{dy} \sin\theta \qquad (5.16)$$

where R is the horizontal load given by

$$R = \alpha WH \qquad (5.17)$$

$V_1$ is the base shear in the blastward column given by

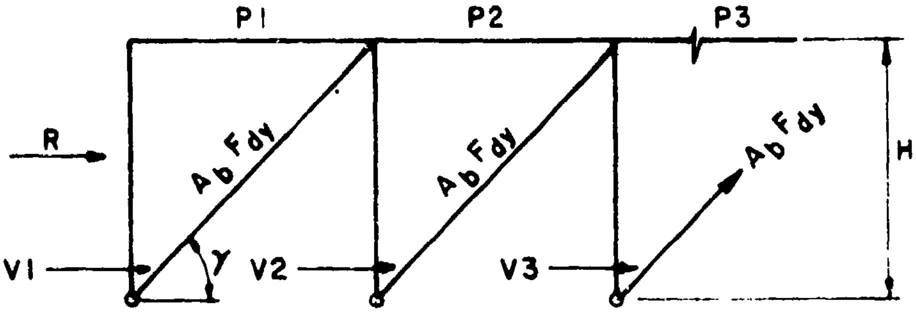
$$V_1 = R/2 + M_p/H$$

and $V_2$ is the shear in each of the remaining columns given by

$$V_2 = R/2n - M_p/nH \qquad (5.19)$$

in which n is the number of bays and $mA_b F_{dy} \sin\theta$ is the sum of the horizontal components of the forces in the braces.

In applying Figure 5.4 to braced frames with non-rigid girder to column connections, the quantity $M_p$ should be set equal to zero. For such cases, the entire horizontal shear is taken by the bracing and the exterior column. For frames with



$$R = \alpha w H$$

$$V1 = R/2 + M_p/H$$

$$P1 = R/2 - M_p/H$$

$$P2 = P1 - V2 - F_H$$

$$F_H = A_b F_{dy} \cos\gamma$$

$$V2 = R/2n - M_p/nH$$

$$P3 = P2 - V2 - F_H$$

$$P_n = P_{n-1} - V2 - F_H$$

Figure 5.4 Column shears and girder axial loads for braced frames.

rigid connections, $M_p$ is simply the design plastic moment obtained from the controlling panel or combined mechanism.

For computing axial loads in the columns the vertical components of the forces in the tension braces should be added to the vertical shear in the roof girders. The vertical component of the force in a brace is given by

$$F_v = A_b F_{dy} \sin\gamma \qquad (5.20)$$

The reactions from the braces will also affect the load on the foundation of the frame and must be included in the design of the footing.

The provisions of Section 5.2.4 on combined bending and axial load for rigid frames apply to braced frames with the exception that the reactions from the tension braces must be taken into account in computing axial loads and shears.

Finally, when the estimated design forces and moments acting on the frame members have been obtained, preliminary member sizes can be determined based upon the material in Chapters 2 through 4.

### 5.3.7 Slenderness Requirements for Diagonal Braces

The slenderness ratio of the bracing should be less than 300 to prevent vibration and "slapping". This design condition can be expressed as

$$r_b \geq L_b/300 \qquad (5.21)$$

in which $r_b$ is the minimum radius of gyration of the bracing member and $L_b$ is its length between points of support. Even though a compression brace is not considered effective in providing resistance, the tension and compression braces should be connected together where they cross. In this manner, $L_b$ for each brace may be taken equal to half of the total length.

### 5.3.8 Preliminary Design Procedures for Frames with Supplementary Bracing.

The preliminary design procedure for frames with supplementary diagonal braces is similar to the procedure described in Section 5.2 for rigid frames and as illustrated by the example problem in Section 7.6. For braced frames with rigid connections, however, the procedure is slightly more involved since it is necessary to assume a value for the brace area in addition to the assumptions for the coefficients C and $C_1$. For frames with non-rigid connections, C and $C_1$ do not appear in the resistance formula for a sidesway mechanism and $A_b$ can be determined directly.

In selecting a trial value of $A_b$ for frames with rigid connections, the minimum brace size will be controlled by the slenderness requirement of Section 5.3.7.

In addition, in each particular application, there will be a limiting value of $A_b$ beyond which there will be no substantial weight savings in the frame members since minimum sizes for the frame members are required based upon the maximum slenderness ratio requirements in Section 4.2. In general, values of $A_b$ of about one to two square inches will result in a substantial increase in the overall resistance for frames with rigid connections. Hence, an assumed brace area in this range is recommended as a starting point. The determination of $C$ and $C_1$ then follows the same procedure as outlined for rigid frames.

# CHAPTER 6

# CONNECTIONS

## 6.1 Introduction

The connections in a steel structure designed in accordance with plastic design concepts must fulfill their function up to the ultimate load capacity of the structure. In order to allow the members to reach their full plastic moments, the connections must be capable of transferring moments, shears and axial loads with sufficient strength, proper stiffness and adequate rotation capacity.

Connections must be designed with consideration of economical fabrication and ease of erection. Connecting devices may be rivets, bolts, welds, screws or various combinations thereof.

## 6.2 Types of Connections

The various connection types generally encountered in steel structures can be classified in the following groups:

A. Primary member connections

1. Corner frame connections
2. Beam to column connections
3. Beam to girder connections
4. Splices
5. Column base connections

B. Secondary member connections

1. Purlin to frame connections
2. Girt to frame connections
3. Bracing connections

C. Attachment of Panels

1. Roof or floor panel connections
2. Wall siding connections.

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Connections in Group A refer to those used in design and construction of the framing of primary members. They generally involve the attachment of hot-rolled sections to one another, either to create specific support conditions or to achieve continuity of a member or the structure. In that respect, connections used in framing may be classified into three groups (corresponding to Type 1, 2 and 3 connections in the AISC Specification) as follows:

(i) Rigid

(ii) Flexible (Non-rigid)

(iii) Semi-rigid

depending upon their degree of restraint which is the ratio of the actual end moment that may be developed to the end moment in a fully fixed-ended beam. Approximately, the degree of restraint is generally considered as over 90 percent for rigid connections, between 20 to 90 percent for semi-rigid connections and below 20 percent for flexible connections.

It should be mentioned that the strength and rotation characteristics of semi-rigid connections are dependent upon the properties of the intermediate connecting elements (angles, plates, tees) and thus, are subject to much variation. Since semi-rigid structural analyses are seldom undertaken due to their great complexity, no further details on semi-rigid connections will be given here.

Connections listed in Group B are used to fasten secondary members such as purlins, girts or bracing members to the primary members of a frame, either directly or by means of auxiliary sections such as angles and tees.

Basic requirements for connections of Groups A and B as well as general guidelines for proper design are presented in Sections 6.3 and 6.4. In addition, dynamic design stresses to be used in the selection and sizing of fastening devices are given in Section 6.5.

Group C refers to connections used to attach elements of the skin or outer shell of an installation as well as floor and wall panels to the supporting skeleton. Connections of this type are distinguished by the fact that they fasten relatively thin sheet material to one another or to heavier rolled sections. Roof decks and wall siding have to withstand during their lifetime (apart from accidental blast loads) exposure to weather, uplift forces, buffeting and vibration due to winds, etc. For this reason, and because of their widespread use, special care should be taken in design to insure their adequate behavior. Some basic requirements for panel connections are presented in Section 6.6.

## 6.3 Requirements for Main Framing Connections

The design requirements for frame connections may be illustrated by considering the behavior of a typical corner connection as shown in Figure 6.1. Two members are joined together without stiffening of the corner web. Assuming that the web thickness is insufficient, the behavior of the connection is represented by Curve 1 which shows that yielding due to shear force starts at a relatively low load. Even though the connection rotates past the required hinge rotation, the plastic moment $M_p$ is not reached. In addition, the elastic deformations are also larger than those assumed by the theoretical design curve.

A second and different connection may behave as indicated by Curve 2. Although the elastic stiffness is satisfactory and the maximum capacity exceeds $M_p$, the connection fails before reaching the required hinge rotation and thus, is unsatisfactory.

These considerations indicate that connections must be designed for strength, stiffness and rotation capacity. They must transmit the required moment, shear and axial load, and develop the plastic moment $M_p$ of the members.

Normally, an examination of a connection to see if it meets the requirements of stiffness will not be necessary. Compared to the total length of the member, the length of the connection is small; and, if the connection is slightly more flexible than the member which it joins, the general effect on the structural behavior is not great. Approximately, the average unit rotation of the connecting zone should not exceed that of an equivalent length of the members being joined.

Of equal importance with the strength of the connection is an adequate reserve of ductility after the plastic moment has been attained. Rotation capacity at plastic hinge locations is essential to the development of the full ultimate load capacity of the stucture.

Some typical framing connections used in blast-resistant structures are shown in Appendix C.

## 6.4 Design of Connections

It is not the intent of this chapter to present procedures and equations for the design of the various types of connections likely to be encountered in the blast-resistant design of a steel structure. Instead, the considerations necessary for a proper design will be outlined. The reader is referred to the many standard texts on plastic design for the procedures and equations.

After completion of the dynamic analysis of the structure, whether by hand calculations or computer analysis, the members are sized for the given loadings. The moments, shears and axial loads at the connections are known. Full recognition must be given to the consideration of rebound or stress reversal in designing the connections. Additionally, in continuous structures, the maximum values of P, M and V may not occur simultaneously and thus, several combinations may have to be considered.

All connecting elements subject to compression should meet the width-thickness ratio requirements as specified in Section 3.3.4.

With rigid connections such as a continuous column-girder intersection, the web area within the boundaries of the connection should meet the shear stress requirements of Section 3.3.3. If the web area is unsatisfactory, diagonal stiffeners or web doubler plates should be provided.

Stiffeners will normally be required to prevent web crippling and preserve flange continuity wherever flange to flange connections occur at columns in a continuous frame. Web crippling must also be checked at points of load application such as beam-girder intersections. In these cases, the requirements of Section 3.3.5 of this report and Sections 1.10.5 and 1.10.10 of the AISC Specification must be considered.

Since bolted joints will develop yield stresses only after slippage of the members has occurred, the use of friction-type bolted connections is not recommended.

## 6.5 Dynamic Design Stresses for Connections

In accordance with Section 2.8 of the AISC Specification, bolts, rivets and welds shall be proportioned to resist the maximum forces using stresses equal to 1.7 times those given in Part 1 of the Specification. Additionally, these stresses are increased by the dynamic increase factor specified in Section 2.2.2; hence,

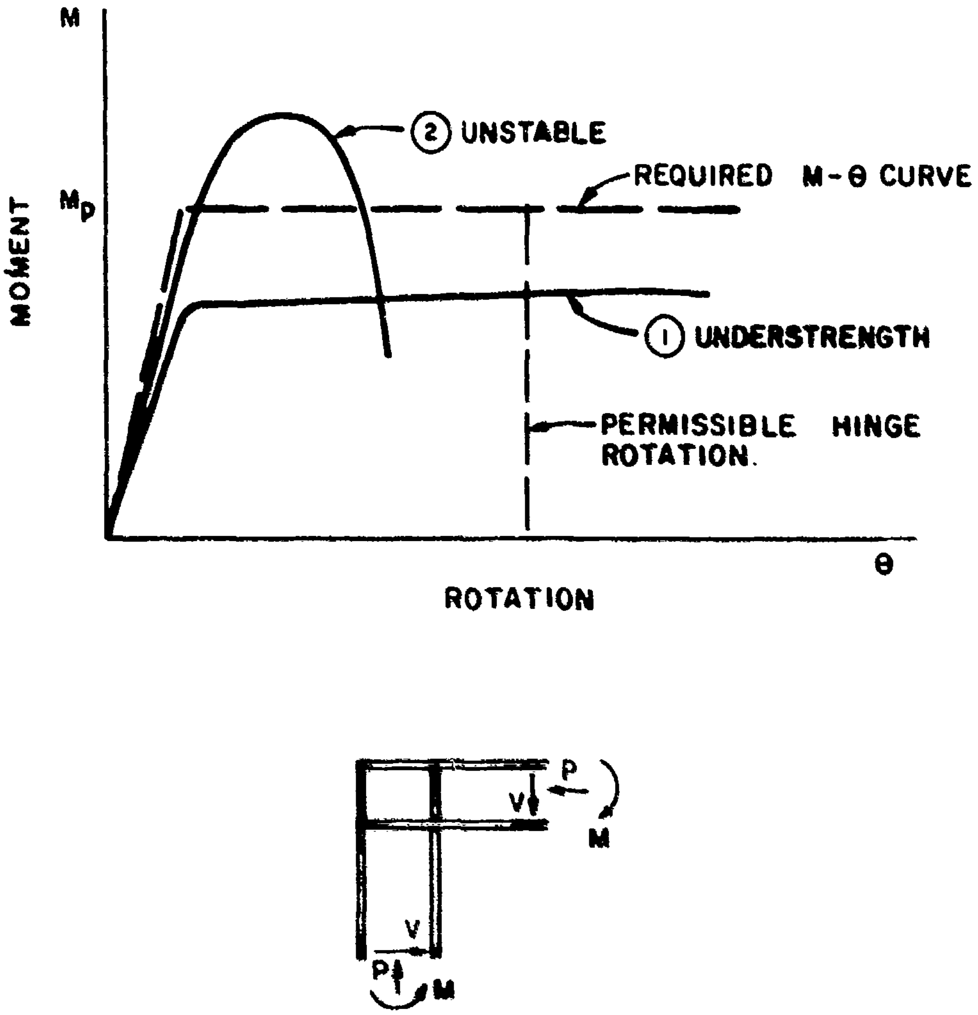
M
② UNSTABLE
REQUIRED M-θ CURVE
$M_p$
MOMENT
① UNDERSTRENGTH
PERMISSIBLE HINGE ROTATION.
θ
ROTATION
P
V
M
V
P
M

Figure 6.1 Corner connection behavior.

$$F_d = 1.7cF_s \qquad (6.1)$$

where

$F_d$ = the maximum dynamic design stress

$c$ = the dynamic increase factor (Table 2.1)

$F_s$ = the allowable static design stress

Rather than compiling new tables for maximum dynamic loads for the various types of connections, the designer will find it advantageous to divide the forces being considered by the factor 1.7c and then to refer to the allowable load tables in Part 1 of the AISC Specification.

## 6.6 Requirements for Floor and Wall Panel Connections

Panel connections, in general, can be divided into two major groups:

a. Panel to panel connections, and

b. Panel to supporting-frame connections.

The first type involves the attachment of relatively light-gage materials to each other such that they act together as an integral unit. The second type is generally used to attach sheet panels to heavier cross-sections.

The most common type of fastener for decking and steel wall panels is the self-tapping screw with or without washer. Even for conventional design and regular wind loading, the screw fasteners have often been the source of local failure by tearing the sheeting material. It is evident that under blast loading and particularly on rebound, screw connectors will be even more vulnerable to this type of failure. Special care should be taken in design to reduce the probability of failure by using oversized washers and/or increased material thickness at the connection itself.

Due to the magnitude of forces involved, special types of connectors, as shown in Fig. 6.2 and Appendix C, will usually be necessary, e.g.:

a. Self-piercing, self-tapping screws of larger diameters with oversized washers

b. Puddle welds or washer plug welds.

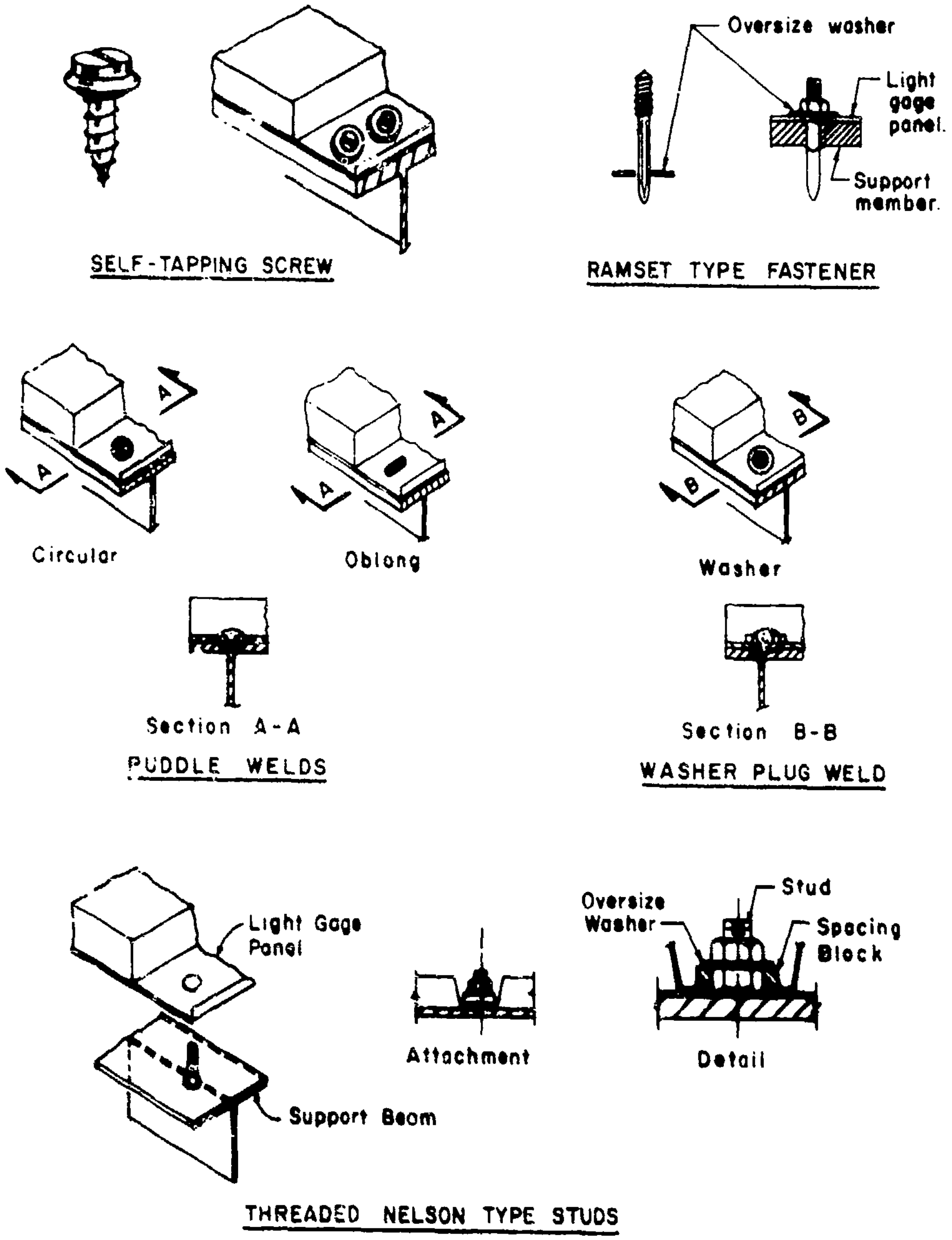
Oversize washer
Light gage panel.
Support member.
SELF-TAPPING SCREW
RAMSET TYPE FASTENER
A
A
Circular
A
A
Oblong
B
B
Washer
Section A-A
PUDDLE WELDS
Section B-B
WASHER PLUG WELD
Light Gage Panel
Support Beam
Attachment
Oversize Washer
Stud
Spacing Block
Detail
THREADED NELSON TYPE STUDS

Figure 6.2 Typical connections for cold-formed steel panels

c. Threaded connectors fired into the elements to be attached.

d. Threaded studs, welded to the supporting members, which fasten the panel by means of a special arrangement of bushing and nut.

Apart from fulfilling their function of cladding and load-resisting surfaces, by carrying loads perpendicular to their surface, floor, roof and wall, steel panels can, when adequately connected, develop substantial resistance to in-plane forces, acting as diaphragms contributing a great deal to the overall stiffness and stability of the structure. As a result, decking connections are in many cases subjected to a combination of shearing forces and pull-out forces. It is to be remembered also that after the panel has deflected under blast loading, the catenary action sustained by the flat sheet of the decking represents an important reserve capacity against total collapse. To allow for such catenary action to take place, connectors and especially end connectors should be made strong enough to withstand the membrane forces that develop.

Finally, the design of these special connections for decking should take into account acceptable construction tolerances, ease of fabrication, simplicity of erection, reliability and cost.

# CHAPTER 7

# DESIGN EXAMPLES

This chapter presents detailed design procedures and numerical examples on the following topics:

1. Flexural elements subjected to pressure-time loading
2. Lateral bracing requirements
3. Cold-formed steel panels
4. Columns and beam-columns
5. Open-web joists
6. Single-story rigid frames
7. Blast doors
8. Unsymmetrical bending

References are made to the appropriate sections in Chapters 1 through 6 of this report and to charts, tables and equations from other design manuals and specifications.

## 7.1 Design of Beams for Pressure-Time Loading

Problem 1: Design a purlin or girt as a flexural member which responds to a pressure-time loading.

Procedure:

Step 1. Establish the design parameters:

a. Pressure-time load

b. Design criteria ($\mu_{max}$ and $\theta_{max}$ for a reusable or non-reusable structure) (Section 2.3.3)

c. Span length, L, beam spacing, b, and support conditions

d. Properties and type of steel used, i.e., $F_y$ and E

Step 2. Determine the equivalent static load w using the following preliminary dynamic load factors as discussed in Section 3.3.2.

$$DLF = \begin{cases} 0.8 \text{ for a non-reusable structure} \\ 1.0 \text{ for a reusable structure} \end{cases}$$

$$w = DLF \times p \times b$$

Step 3. Using the appropriate resistance formula from Table 3.1 and the equivalent static load derived in Step 2, determine $M_p$.

Step 4. Select a member size using Equation 3.2 or 3.3. Check the local buckling criteria of Section 3.3.4 for the member chosen.

Step 5. Determine the mass m including the weight of the decking over a distance center-to-center of purlins or girts, and the weight of the members.

Step 6. Calculate the equivalent mass $M_e$ using Table 6-1 of TM 5-1300.

Step 7. Determine the equivalent elastic stiffness $K_E$ from Table 3.1.

Step 8. Calculate the natural period of vibration, $T_N$, using Equation 3.10.

Step 9. Determine the total resistance, $R_u$, and peak pressure load, B. Enter Figure 6-7 of TM 5-1300 with the ratios $T/T_N$ and $B/R_u$ in order to establish the ductility ratio $\mu$. Compare with criteria in Step 1.

Step 10. Calculate the equivalent elastic deflection $X_E$ as given by the equation

$$X_E = R_u/K_E$$

and establish the maximum deflection $X_m$ given by

$$X_m = \mu X_E$$

Compute the corresponding member end rotation. Compare $\theta$ with the criteria in Section 2.3.3.

$$\tan\theta = X_m/(L/2)$$

Step 11. Check for shear using Equation 3.11 and Table 3.3.

Step 12. If a different member size is required, repeat Steps 2 through 11 by selecting a new dynamic load factor.

Example 1: Design of a beam for pressure-time loading.

Required: Design a reusable simply-supported beam in a low to intermediate pressure range.

Step 1. Given:

a. Pressure-time loading (Figure 7.1)

b. Criteria: Reusable structure, maximum ductility ratio = 3, maximum end rotation = 1°, whichever governs

c. Structural configuration (Figure 7.1)

d. $F_y$ = 36 ksi, E = 30 x $10^3$ ksi, A36 steel

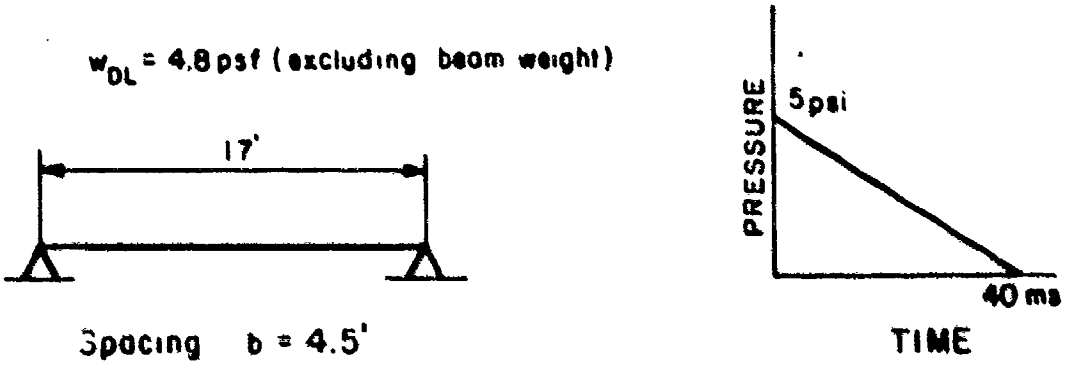

Figure 7.1 Beam configuration and loading, Example 1.

Step 2. Determine the equivalent static load (i.e., required resistance).

DLF = 1.0 for a reusable structure (Section 3.3.2)

$$w = 1.0 \times 5 \times 4.5 \times 144/1000 = 3.24 \text{ k/ft}$$

Step 3. Determine required $M_p$.

$$M_p = \frac{wL^2}{8} = \frac{3.24 \times 17^2}{8} = 117 \text{ k-ft} \qquad \text{(Table 3.1)}$$

Step 4. Select a member.

$$(S + Z) = \frac{2M_p}{F_{dy}} = \frac{2 \times 117 \times 12}{39.6} = 71.0 \text{ in}^3 \qquad \text{(Equation 3.2)}$$

where $F_{dy} = cF_y = 1.1 \times 36 = 39.6$ ksi (Table 2.1)

Select W12 x 27, $S = 34.2 \text{ in}^3$ $I = 204 \text{ in}^4$

$Z = 38.0 \text{ in}^3$

$S + Z = 72.2 \text{ in}^3$

$M_p = (72.2 \times 39.6)/(2 \times 12) = 119$ k-ft

Check local buckling criteria.

$d/t_w = 50.5 < 412/\sqrt{F_y} = 68.7$ O.K. (Equation 3.12)

$b_f/2t_f = 8.12 < 8.5$ O.K. (Section 3.3.4)

Step 5. Calculate M.

$$M = \frac{wL}{g} = \frac{[(4.5 \times 4.8) + 27](17 \times 10^6)}{32.2 \times 1000}$$

$$= 25{,}600 \text{ (k-ms}^2\text{)/ft}$$

Step 6. Calculate the effective mass, $M_e$, for a response in the elasto-plastic range.

$$K_{LM} = (0.79 + 0.66)/2 = 1.45/2 = 0.725$$

(Table 6-1, TM 5-1300)

$$M_e = 0.725 \times 25{,}600 = 18{,}560 \text{ k-ms}^2/\text{ft}$$

Step 7. Determine $K_E$.

$$K_E = \frac{384\ EI}{5L^3} = \frac{384 \times 30 \times 10^3 \times 204}{5 \times 17^3 \times 144}$$

$$= 664 \text{ k/ft}$$

(Table 3.1)

Step 8. Calculate $T_N$.

$$T_N = 2\pi\sqrt{M_e/K_E} = 2\pi\sqrt{18{,}560/664}$$

$$= 34 \text{ ms}$$

(Equation 3.10)

Step 9. Establish the ductility ratio $\mu$ and compare with the criteria.

$$T/T_N = 40/34 = 1.18$$

$$B = p \times L \times b = \frac{5 \times 17 \times 4.5 \times 144}{1000}$$

$$= 49 \text{ kips}$$

$$R_u = 8M_p/L = (8 \times 119)/19 = 50 \text{ kips}$$

$$B/R_u = 49/50 = 0.98$$

From Figure 6-7 of TM 5-1300,

$$\mu = X_m/X_E = 2.05 < 3 \qquad \text{O.K.}$$

Step 10. Determine $X_E$.

$$X_E = R_u/K_E = (50 \times 12)/664 = 0.90 \text{ inch}$$

Find $X_m$.

$$X_m = \mu X_E = 2.05 \times 0.90 = 1.845 \text{ inches}$$

Find end rotation, $\theta$.

$$\tan\theta = X_m/(L/2) = 1.845/(8.5 \times 12) = 0.0181$$

$$\theta \approx 1^\circ \qquad \text{O.K.}$$

(Section 2.3.3)

$$K_{LM} = (0.79 + 0.66)/2 = 1.45/2 = 0.725$$
(Table 6-1, TM 5-1300)

$$M_e = 0.725 \times 25{,}600 = 18{,}560 \text{ k-ms}^2/\text{ft}$$

Step 7. Determine $K_E$.

$$K_E = \frac{384\ EI}{5L^3} = \frac{384 \times 30 \times 10^3 \times 204}{5 \times 17^3 \times 144}$$

$$= 664 \text{ k/ft}$$
(Table 3.1)

Step 8. Calculate $T_N$.

$$T_N = 2\pi\sqrt{M_e/K_E} = 2\pi\sqrt{18{,}560/664}$$

$$= 34 \text{ ms}$$
(Equation 3.10)

Step 9. Establish the ductility ratio $\mu$ and compare with the criteria.

$$T/T_N = 40/34 = 1.18$$

$$B = p \times L \times b = \frac{5 \times 17 \times 4.5 \times 144}{1000}$$

$$= 49 \text{ kips}$$

$$R_u = 8M_p/L = (8 \times 119)/19 = 50 \text{ kips}$$

$$B/R_u = 49/50 = 0.98$$

From Figure 6-7 of TM 5-1300,

$$\mu = X_m/X_E = 2.05 < 3 \quad \text{O.K.}$$

Step 10. Determine $X_E$.

$$X_E = R_u/K_E = (50 \times 12)/664 = 0.90 \text{ inch}$$

Find $X_m$.

$$X_m = \mu X_E = 2.05 \times 0.90 = 1.845 \text{ inches}$$

Find end rotation, $\theta$.

$$\tan\theta = X_m/(L/2) = 1.845/(8.5 \times 12) = 0.0181$$

$$\theta = 1^\circ \qquad \text{O.K.}$$
(Section 2.3.3)

Step 11. Check for shear.

Dynamic yield stress in shear

$F_{dv} = 0.55F_{dy} = 0.55 \times 39.6 = 21.8$ ksi (Equation 2.3)

Ultimate shear capacity

$V_p = F_{dv} \times A_w = 21.8 \times 0.24 \times 12$

$= 62.7$ kips (Equation 3.11)

Maximum support shear

$V_s = r_u \times L/2 = R_u/2 = 50/2 = 25.0$ kips

$V_p > V_s$ O.K.

## 7.2 Spacing of Lateral Bracing

Problem 2: Investigate the adequacy of the lateral bracing specified for a flexural member.

The design procedure for determining the maximum permissible spacing of lateral bracing is essentially a trial and error procedure if the unbraced length is determined by the consideration of lateral torsional buckling only. However, in practical design, the unbraced length is usually fixed by the spacing of purlins and girts and then must be investigated for lateral torsional buckling.

Procedure:

Step 1. Establish design parameters.

a. Bending moment diagram obtained from a design analysis

b. Unbraced length, $l$, and radius of gyration of the member, $r_y$, about its weak axis

c. Dynamic yield strength, $F_{dy}$ (Section 2.2.3)

Step 2. From the moment diagram, find the end moment ratio, $M/M_p$, for each segment of the beam between points of bracing.

(Note that the end moment ratio is positive when the segment is bent in reverse curvature and negative when bent in single curvature).

Step 3. Compute the maximum permissible unbraced length, $l_{cr}$, using Equation 3.14 or 3.15, as applicable. Since the spacing of purlins and girts is usually uniform, the particular unbraced length that must be investigated in a design will be the one with the largest moment ratio. The spacing of bracing in non-yielded segments of a member should be checked against the requirements of Section 1.5.1.4.6a of the AISC Specification (see Section 3.3.6).

Step 4. The actual length of the segment being investigated should be less than or equal to $l_{cr}$.

Example 2: Spacing of Lateral Bracing

Required: Investigate the unbraced lengths shown for the W10 x 21 beam in Figure 7.2.

Step 1. Given:

a. Bending moment diagram shown in Figure 7.2

b. Unbraced length (each segment) = 44 inches
$r_y$ = 1.32 inches

c. Dynamic yield strength = 39.6 ksi

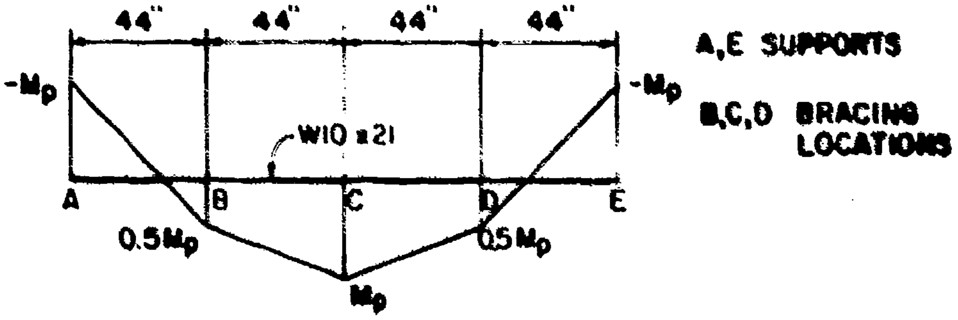

Figure 7.2 Bending moment diagram, Example 2.

Step 2. The moment ratio is -0.5 for segments BC and CD (single curvature) and 0.5 for segments AB and DE (double curvature).

Step 3. Determine $l_{cr}$.

$$l_{cr} = \frac{1375.0 \times 1.32}{39.6} = 46.0 \text{ inches} \qquad \text{(Equation 3.15)}$$

By inspection, Equation 3.15 results in a larger unbraced length than Equation 3.14.

Step 4. Since the actual unbraced length is less than 46 inches, the spacing of the bracing is adequate.

## 7.3 Design of Cold-Formed, Light-Gage Steel Panels Subjected to Pressure-Time Loading

Problem 3: Design a roof deck as a flexural member which responds to pressure-time transverse loading.

Procedure:

Step 1. Establish the design parameters:

a. Pressure-time loading

b. Design criteria ($\mu_{max}$ and $\theta_{max}$ for either a reusable or non-reusable cold-formed panel) (Section 2.3.3)

c. Span length and support conditions

d. Mechanical properties of steel

Step 2. Determine an equivalent uniformly distributed static load for 1-ft width of panel, using the following preliminary dynamic load factors.

|  | Reusable | Non-Reusable |
|---|---|---|
| DLF | 1.65 | 1.40 |

These load factors are based on an average value of $T/T_N = 10.0$ and the design ductility ratios recommended in Equation 3.27. They are derived using Figure 6-7 of TM 5-1300.

Equivalent static load $w = DLF \times 1.1 \times p \times b$ where the 1.1 increase factor is outlined in Section 3.7.2 and $b = 1$ ft.

Step 3. Using the equivalent static load derived in Step 2, determine the ultimate moment capacity (positive and negative depending on support conditions). (Section 3.7.2)

Step 4. Determine required section moduli using Equation 3.22 or 3.23.

Select a panel.

Step 5. Determine actual section properties of the panel: $S^+$, $S^-$, $I$, $m = w/g$ (for 1-ft width of a panel).

Step 6. Compute $r_u$, the maximum unit resistance per 1-ft width of panel using Equation 3.24 or 3.25.

Step 7. Determine the equivalent elastic stiffness, $K_E = r_u L / X_E$ using Equation 3.26.

Step 8. Compute the natural period of vibration

$$T_N = 2\pi\sqrt{0.74mL/K_E} \qquad \text{(Equation 3.29)}$$

Step 9. Calculate $B/r_u$ and $T/T_N$. Enter Figure 6-7 of TM 5-1300 with the ratios $B/r_u$ and $T/T_N$ to establish the actual ductility ratio $\mu$.

Compare $\mu$ with the criteria of Section 2.3.3. If $\mu$ is larger than the criteria value, repeat Steps 4 to 9.

Step 10. Compute the equivalent elastic deflection $X_E$ using $X_E = r_u L/K_E$.

Evaluate the maximum deflection, $X_m = \mu X_E$.

Determine maximum panel end rotation,

$$\tan\theta = X_m/(L/2).$$

Compare $\theta$ with the criteria of Section 2.3.3. If $\theta$ is larger than specified in the criteria, select another panel and repeat Steps 5 to 10.

Step 11. Check resistance in rebound using chart in Figure 3.9.

Step 12. Check panel for maximum resistance in shear by applying the criteria relative to:

a. simple shear, Table 3.4(a)

b. combined bending and shear, Table 3.4(b)

c. web crippling, Figures 3.10(a) and 3.10(b).

If the panel is inadequate in shear, select a new member and repeat Steps 4 to 12.

Example 3: Design of a roof deck for pressure-time loading.

Required: Design a reusable continuous cold-formed steel panel in a low to intermediate pressure range.

Step 1. Given:

a. Pressure-time loading (Figure 7.3)

b. Criteria:

maximum ductility ratio, $\mu_{max} = 1.25$

maximum rotation, $\theta_{max} = 0.9^{\circ}$

c. Structural configuration (Figure 7.3)

d. Steel A446 Grade a, $E = 30 \times 10^6$ psi

$F_y = 33{,}000$ psi

$c = 1.1$

Step 2. Determine the equivalent static load.

DLF = 1.65 (reusable)

$w = 1.1 \times DLF \times p \times b$

$= 1.1 \times 1.65 \times 4.30 \times 12 \times 12 = 1124$ lb/ft

= equivalent static load for 1-ft width of panel.

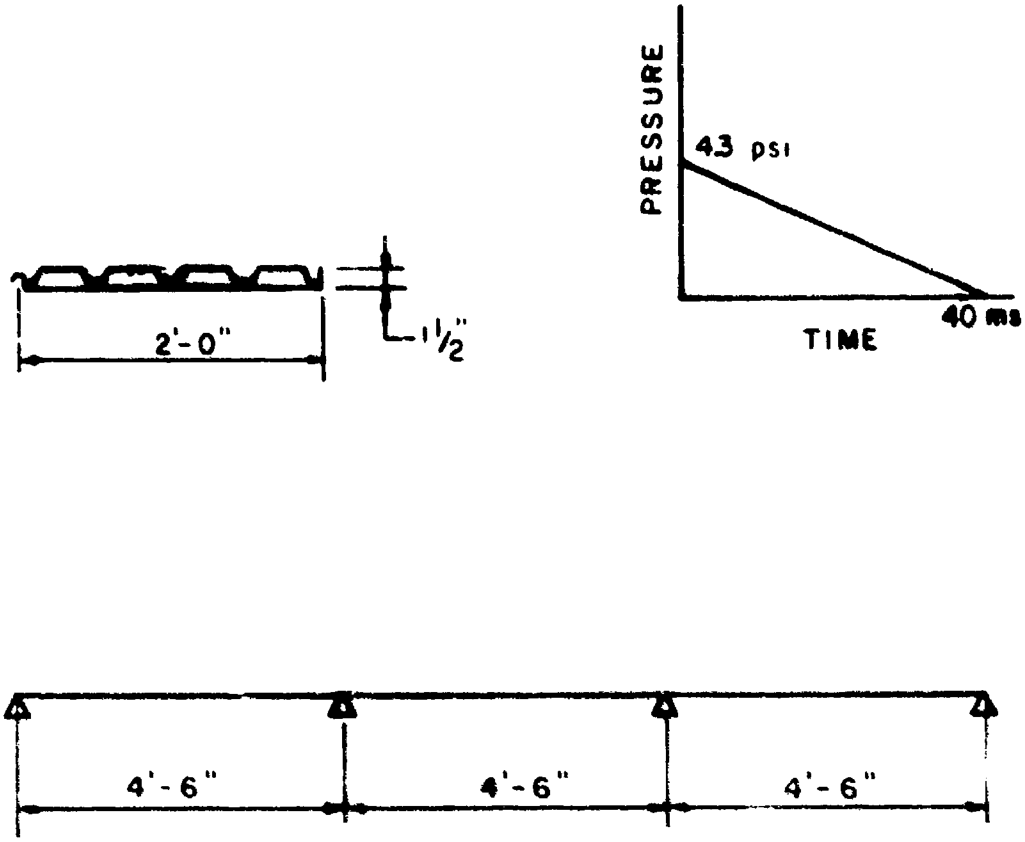
2'-0"
1½"
PRESSURE
43 psi
TIME
40 ms
4'-6"
4'-6"
4'-6"

Figure 7.3 Roof decking configuration and loading, Example 3.

Step 3. Determine required ultimate moment capacities.

$$M_{up} = \frac{wL^2}{12.2} = \frac{1124 \times (4.5)^2}{12.2} = 1{,}866 \text{ lb-ft} \qquad \text{(Equation 3.25)}$$

$$M_{un} = \frac{wL^2}{10.1} = \frac{1124 \times (4.5)^2}{10.1} = 2{,}254 \text{ lb-ft} \qquad \text{(Equation 3.25)}$$

Step 4. Determine required section moduli.

$$F_{dy} = 1.10 \times 33{,}000 = 36{,}300 \text{ psi} \qquad \text{(Equation 3.21)}$$

$$S^{+} = \frac{1866 \times 12}{36{,}300} = .617 \text{ in}^3 \qquad \text{(Equation 3.22)}$$

$$S^{-} = \frac{2254 \times 12}{36{,}300} = .746 \text{ in}^3 \qquad \text{(Equation 3.23)}$$

Select a 1-1/2 inch deep panel 16-16.
(hat section 16 gage, flat sheet 16 gage)

Step 5. Determine actual section properties.

$S^{+} = .654 \text{ in}^3$

$S^{-} = .745 \text{ in}^3$

$I = .763 \text{ in}^4$

$w = 5.8$ psf

Step 6. Compute maximum unit resistance $r_u$.

$$M_{up} = \frac{36{,}300 \times .654}{12} = 1{,}978 \text{ lb-ft} \qquad \text{(Equation 3.22)}$$

$$M_{un} = \frac{36{,}300 \times .745}{12} = 2{,}254 \text{ lb-ft} \qquad \text{(Equation 3.23)}$$

$$r_u^{+} = \frac{1978 \times 12.2}{(4.5)^2} = 1{,}192 \text{ lb/ft} \qquad \text{(Equation 3.25)}$$

$$r_u^{-} = \frac{2254 \times 10.1}{(4.5)^2} = 1{,}124 \text{ lb/ft} \qquad \text{(Equation 3.25)}$$

Maximum resistance $r_u = 1{,}124$ lb/ft, governed by capacity at support.

Step 7. Determine equivalent static stiffness.

$$K_E = \frac{r_u L}{X_E} = \frac{EI_{eq} \times r_u \times L}{0.0062 \times r_u \times L^4} \quad \text{(Equation 3.26)}$$

$$= \frac{EI_{eq}}{0.0062 \times L^3}, \; I_{eq} = 0.75I$$

$$= \frac{30 \times 106 \times .75 \times .763}{0.0062 \times (4.5)^3 \times 144} = 211{,}000 \text{ lb/ft}$$

Step 8. Compute the natural period of vibration for the 1-ft width of panel.

$$mL = \frac{w}{g} = \frac{5.8 \times 10^6 \times 4.5}{32.2} = 0.81 \times 10^6 \; \frac{\text{lb-ms}^2}{\text{ft}}$$

$$T_N = 2\pi\sqrt{(0.74 \times 0.81 \times 10^6)/211{,}000} = 10.6 \text{ ms}$$

Step 9. Calculate $B/r_u$, $T/T_N$.

$$B = 1.1 \times p \times b$$

$$= 1.1 \times 4.30 \times 12 \times 12 = 681 \text{ lb/ft}$$

$$B/r_u = 681/1124 = 0.606$$

$$T/T_N = 40/10.6 = 3.78$$

Entering Figure 6-7 in TM 5-1300 with these values,

$\mu = 1.20 < 1.25$ O.K.

Step 10. Check maximum deflection and rotation.

$$X_E = r_u L/K_E$$

$$X_m = \mu X_E = 1.20 \times \frac{1{,}124}{46{,}879} = .0287 \text{ ft}$$

$$\tan\theta = X_m/(L/2) = .0287/2.25 = 0.01275$$

$\theta \approx 0.73^\circ < 0.9^\circ$ O.K.

Step 11. Check resistance in rebound.

From chart, Figure 3.9, required $\bar{r}/r_u = 0.50$

Available maximum elastic resistance in rebound

$$(\bar{r}/r_u)_{actual} = .654/.745$$

$$= .875 > 0.50 \qquad \text{O.K.}$$

Step 12. Check resistance in shear.

a. Interior support (combined shear and bending)

Determine dynamic shear capacity of a 1-ft width of panel:

$$h = (1.500 - 2t) \text{ inches, } t = .060$$

$$= 1.5000 - 0.120 = 1.380 \text{ inches}$$

$$h/t = 1.380/0.060 = 23 \approx 20$$

$$F_{dv} = 9.0 \text{ ksi} \qquad \text{(Table 3.4b)}$$

Total web area for 1-ft width of panel

$$(8 \times h \times t)/2 = 4 \times 1.380 \times .060$$

$$= 0.3312 \text{ in}^2$$

$$V_u = 0.3312 \times 9{,}000 = 2{,}980 \text{ lb}$$

Determine maximum dynamic shear force:

The maximum shear at an interior support of a continuous panel using limit design is:

$$V_{max} = 0.55 \times r_u \times L$$

$$= 0.55 \times 1.124 \times 4.5$$

$$= 2{,}782 \text{ lb} < 2{,}980 \text{ lb} \qquad \text{O.K.}$$

b. End support (simple shear)

Determine dynamic shear capacity of a 1-ft width of panel:

For $h/t \leq 63$, $F_{dv} = 0.50F_{dy} = 0.5 \times 36.3$

$= 18.15$ ksi

(Table 3.4a)

$V_u = 0.3312 \times 18{,}150 = 6{,}011$ lb

Determine maximum dynamic shear force:

The maximum shear at an end support of a continuous panel using limit design is

$V_{max} = 0.45 \times r_u \times L = 0.45 \times 1{,}124 \times 4.5$

$= 2{,}276 \text{ lb} < 6{,}011 \text{ lb}$ O.K.

c. Web crippling

End support (N = 2-1/2 inches)

$Q_u = 1{,}400 \times 4 = 5{,}600 \text{ lbs} > 2{,}276$ O.K.
(Figure 3.10a)

Interior support (N = 5 inches)

$Q_u = (3{,}500 \times 4)/2 = 7{,}000 \text{ lbs} > 2{,}782$ O.K.
(Figure 3.10b)

## 7.4 Design of Columns and Beam-Columns

Problem 4: Design a column or beam-column for axial load combined with bending about the strong axis.

Procedure:

Step 1. Establish design parameters.

a. Bending moment M, axial load P and shear V are obtained from either a preliminary design analysis or a computer analysis.

b. Span length $l$ and unbraced lengths $l_x$ and $l_y$.

c. Properties of structural steel:

Minimum yield strength $F_y$

Dynamic increase factor c (Table 2.1)

Dynamic yield strength $F_{dy}$ (Section 2.2.3)

Step 2. Select a preliminary member size with a section modulus S such that

$$S \geq M/F_{dy}$$

and $b_f/2t_f$ complies with the structural steel being used. (Section 3.3.4)

Step 3. Calculate $P_y$ (Section 3.3.4) and the ratio $P/P_y$. Using either Equation 3.12 or 3.13, determine the maximum allowable $d/t_w$ ratio and compare it to that of the section chosen. If the allowable $d/t_w$ ratio is less than that of the trial section, choose a new trial section.

Step 4. Check the shear capacity of the web. Determine the web area $A_w$ (Section 3.3.3) and the allowable dynamic shear stress $F_{dv}$ (Section 2.2.3). Calculate the web shear capacity $V_p$ (Equation 3.11) and compare to the design shear V. If inadequate, choose a new trial section and return to Step 3.

Step 5. Determine the radii of gyration, $r_x$ and $r_y$, and plastic section modulus, Z, of the trial section from the AISC Handbook.

Step 6. Calculate the following quantities using the various design parameters:

a. Equivalent plastic resisting moment

$$M_p = F_{dy}Z$$

b. Effective slenderness ratios $Kl_x/r_x$ and $Kl_y/r_y$. For the effective length factor K, see Section 1.8 of the Commentary on the AISC Specification and Section 4.3.

c. Allowable axial stress $F_a$ corresponding to the larger value of $Kl/r$.

d. Allowable moment $M_m$ from Equation 4.6 or 4.7.

e. $F'_e$ and "Euler" buckling load $P_e$ (Section 4.2).

f. Plastic axial load $P_p$ (Section 4.2) and ultimate axial load $P_u$ (Equation 4.2).

g. Coefficient $C_m$ (Section 1.6.1 AISC Specification).

Step 7. Using the quantities obtained in Step 6 and the applied moment M and axial load P, check the interaction formulas (Equations 4.4 and 4.5). Both formulas must be satisfied for the trial section to be adequate.

Example 4(a): Design of a roof girder as a beam-column.

Required: Design a fixed-ended roof girder in a framed structure for combined bending and axial load.

Step 1. Given:

a. Preliminary computer analysis gives the following values for design:

$M_x$ = 115 ft-kips

$M_y$ = 0

P = 53.5 kips

V = 15.1 kips

b. Span length $l$ = 17'-0"

Unbraced lengths $l_x$ = 17'-0" and $l_y$ = 17'-0"

c. A36 structural steel

$F_y$ = 36 ksi

c = 1.10 (Table 2.1)

$F_{dy} = cF_y$ = 1.10(36) = 39.6 ksi (Section 2.2.3)

Step 2.

$S = M_x/F_{dy} = 115(12)/39.6 = 34.8 \text{ in}^3$

Try W12x36 ($S = 46 \text{ in}^3$)

$A = 10.6 \text{ in}^2$ $\qquad d/t_w = 40.1$

$b_f/2t_f = 6.08 < 8.5$ O.K. (Section 3.3.4)

Step 3.

$P_y = AF_y = 10.6(36) = 382$ kips (Section 3.3.4)

$P/P_y = 53.5/382 = 0.140 < 0.27$

$d/t_w = (412/\sqrt{F_y})[1 - 1.4(P/P_y)]$ (Equation 3.12)

$= (412/\sqrt{36})[1 - 1.4(0.140)]$

$= 55.2 > 40.1$ O.K.

Step 4.

$V_p = F_{dv}A_w$ (Equation 3.11)

$F_{dv} = 0.55F_{dy} = 0.55(39.6) = 21.8$ ksi (Section 2.2.3)

$A_w = t_w(d - 2t_f) = 0.305[12.24 - 2(0.540)]$

$= 3.40 \text{ in}^2$ (Section 3.3.3)

$V_p = 21.8(3.40) = 74.1 \text{ kips} > 15.1 \text{ kips}$ O.K.

Step 5.

$r_x = 5.15$ in.

$r_y = 1.55$ in.

$Z = 51.6 \text{ in}^3$ (AISC Manual)

Step 6.

$$M_{px} = F_{dy} \times Z_x$$ (Section 4.2)

$$= 39.6 \times 51.6 \times 1/12 = 170.3 \text{ ft-kips}$$

$$K = 0.75$$ (Section 1.8, Commentary on AISC Specification)

$$Kl_x/r_x = [0.75(17)12]/5.15 = 30$$

$$Kl_y/r_y = [0.75(17)12]/1.55 = 99$$

$$F_a = 13.10 \text{ ksi for } Kl_y/r_y = 99 \text{ and } F_y = 36 \text{ ksi}$$ (Appendix A, AISC Specification)

$$1.10(13.10) = 14.41 \text{ ksi for } F_{dy} = 39.6 \text{ ksi}$$

$$M_{mx} = \left[1.07 - \frac{(l/r_y)\sqrt{F_{dy}}}{3{,}160}\right]M_{px} \le M_{px}$$ (Equation 4.7)

$$= \left[1.07 - \frac{(204/1.55)\sqrt{39.6}}{3{,}160}\right]170.3$$

$$= 137.6 < 170.3 \text{ ft-kips}$$

$$F'_{ex} = \frac{12\pi^2 E}{23(Kl_b/r_x)^2} = \frac{12\pi^2(29{,}000)}{23(30)^2} = 165.7 \text{ ksi}$$ (Section 4.2)

$$P_{ex} = \frac{23AF'_{ex}}{12} = \frac{23(10.6)165.7}{12} = 3{,}360 \text{ kips}$$ (Section 4.2)

$$P_p = F_{dy}A = 39.6(10.6) = 420 \text{ kips}$$ (Section 4.2)

$$P_u = 1.7AF_a = 1.7(10.6)14.41 = 260 \text{ kips}$$ (Equation 4.2)

$$C_{mx} = 0.85$$ (Section 1.6.1, AISC Specification)

Step 7.

$$\frac{P}{P_u} + \frac{C_{mx}M_x}{(1 - P/P_{ex})M_{mx}} + \frac{C_{my}M_y}{(1 - P/P_{ey})M_{my}} \leq 1$$

(Equation 4.4)

$$= \frac{53.5}{260} + \frac{0.85(115)}{(1 - 53.5/3360)137.6}$$

$$= 0.206 + 0.722 = 0.928 < 1 \qquad \text{O.K.}$$

$$\frac{P}{P_p} + \frac{M_x}{1.18M_{px}} + \frac{M_y}{1.18M_{py}} \leq 1$$

(Equation 4.5)

$$= \frac{53.5}{420} + \frac{115}{1.18(161)} = 0.127 + 0.572$$

$$= 0.699 < 1 \qquad \text{O.K.}$$

Trial section meets the requirements of Chapter 4.

Example 4(b): Design of column.

Required: Design an exterior fixed-pinned column in a framed structure for biaxial bending plus axial load.

Step 1. Given:

a. Preliminary design analysis of a particular column gives the following values at a critical section:

$M_x$ = 311 ft-kips

$M_y$ = 34 ft-kips

P = 76 kips

V = 54 kips

b. Span length $l$ = 17'-3"

Unbraced lengths $l_x$ = 17'-3" and $l_y$ = 4'-0" (laterally supported by wall girts)

c. A36 structural steel

$F_y = 36$ ksi

$c = 1.10$ (Table 2.1)

$F_{dy} = cF_y = 1.10(36)$

$= 39.6$ ksi (Section 2.2.3)

Step 2.

$S = M_x/F_{dy} = 311(12)/39.6 = 94.2 \text{ in}^3$

Try W14 x 78 ($S = 121 \text{ in}^3$)

$A = 22.9 \text{ in}^2$ $d/t_w = 32.9$

$b_f/2t_f = 8.36 < 8.5$ O.K. (Section 3.3.4)

Step 3.

$P_y = AF_y = 22.9(36) = 824$ kips (Section 3.3.4)

$P/P_y = 76/824 = 0.0922 < 0.27$

$d/t_w = (412/\sqrt{F_y})[1 - 1.4(P/P_y)]$ (Equation 3.12)

$= (412/\sqrt{36})[1 - 1.4(0.0922)]$

$= 59.9 > 32.9$ O.K.

Step 4.

$V_p = F_{dv}A_w$ (Equation 3.11)

$F_{dv} = 0.55F_{dy} = 0.55(39.6)$

$= 21.8$ ksi (Section 2.2.3)

$A_w = t_w(d - 2t_f) = 0.428[14.06 - 2(0.718)]$

$= 5.40 \text{ in}^2$ (Section 3.3.3)

$V_p = 21.8(5.40) = 117.7 \text{ kips} > 54 \text{ kips}$ O.K.

Step 5.

$r_x = 6.09$ inches

$r_y = 3.00$ inches

$Z_x = 134 \text{ in}^3$

$Z_y = 52.4 \text{ in}^3$ (AISC Manual)

Step 6.

$M_p = F_{dy}Z$ (Section 4.2)

$M_{px} = 39.6 \times 134 \times 1/12 = 442$ ft-kips

$M_{py} = 39.6 \times 52.4 \times 1/12 = 173$ ft-kips

Use $K = 1.5$ (Section 4.3)

$$\frac{Kl_x}{r_x} = \frac{1.5(17.25)12}{6.09} = 51$$

$$\frac{Kl_y}{r_y} = \frac{1.5(4.00)12}{3.00} = 24$$

$F_a = 18.26$ ksi for $Kl_x/r_x = 51$ and $F_y = 36$ ksi

$1.10(18.26) = 20.09$ ksi for $F_{dy} = 39.6$ ksi

$M_{mx} = M_{px} = 442$ ft-kips

$M_{my} = M_{py} = 173$ ft-kips (Equation 4.6)

$$F'_{ex} = \frac{12\pi^2 E}{23(Kl_b/r_x)^2} = \frac{12\pi^2(29,000)}{23(51)^2} = 57.3 \text{ ksi}$$ (Section 4.2)

$$F'_{ey} = \frac{12\pi^2 E}{23(Kl_b/r_y)^2} = \frac{12\pi^2(29,000)}{23(24)^2} = 259 \text{ ksi}$$ (Section 4.2)

$$P_{ex} = \frac{23AF'_{ex}}{12} = \frac{23(22.9)57.3}{12} = 2,510 \text{ kips}$$ (Section 4.2)

$$P_{ey} = \frac{23AF'_{ey}}{12} = \frac{23(22.9)259}{12} = 11,370 \text{ kips}$$ (Section 4.2)

$$P_p = F_{dy}A = 39.6(22.9) = 907 \text{ kips}$$

$$P_u = 1.7AF_a = 1.7(22.9)20.09 = 783 \text{ kips}$$

$$C_{mx} = C_{my} = 0.85$$ (Section 1.6.1, AISC Specification)

Step 7.

$$\frac{P}{P_u} + \frac{C_{mx}M_x}{(1 - P/P_{ex})M_{mx}} + \frac{C_{my}M_y}{(1 - P/P_{ey})M_{my}} \le 1$$

(Equation 4.4)

$$\frac{76}{783} + \frac{0.85(311)}{(1 - 76/2510)442} + \frac{0.85(34)}{(1 - 76/11,370)173} =$$

$$0.097 + 0.617 + 0.168 = 0.882 < 1 \quad \text{O.K.}$$

$$P/P_p + M_x/(1.18M_{px}) + M_y/(1.18M_{py}) \le 1$$

(Equation 4.5)

$$76/907 + 311/[1.18(442)] + 34/[1.18(173)] =$$

$$0.084 + 0.596 + 0.166 = 0.846 < 1 \quad \text{O.K.}$$

Trial section meets the requirements of Chapter 4.

## 7.5 Design of Open-Web Steel Joists

Problem 5: Analysis or design of an open-web joist subjected to a pressure-time loading.

Procedure:

Step 1. Establish design parameters.

a. Pressure-time curve

b. Clear span length and joist spacing

c. Minimum yield stress $F_y$ for chord and web members

Dynamic increase factor, (Table 2.1)

d. Design ductility ratio $\mu$ and maximum end rotation for a reusable or non-reusable structure (Section 2.3.3)

Step 2. Select a preliminary joist size as follows:

a. Assume a dynamic load factor (Section 3.3.2)

b. Compute equivalent static load on joist due to blast overpressure

$$w_1 = DLF \times p \times b$$

(Dead load of joist and decking not included)

c. Equivalent service live load on joist

$$w_2 = w_1/1.87 \qquad \text{(Section 3.8)}$$

d. From "Standard Load Tables" adopted by the Steel Joist Institute and AISC, select a joist for the given span and the structural steel being used, with a safe service load (dead load of joist and decking excluded) equal to or greater than $w_2$.

Check whether ultimate capacity of joist is controlled by flexure or by shear.

Step 3. Find the resistance of the joist by multiplying the safe service load by 1.87.

Step 4. Calculate the stiffness of the joist, $K_E$, using Table 3.1.

Determine the equivalent elastic deflection $X_E$ given by,

$$X_E = r_u L/K_E$$

Step 5. Determine the effective mass using the weight of the joist with its tributary area of decking, and the corresponding load-mass factor given in Table 6-1 of TM 5-1300.

Calculate the natural period of vibration, $T_N$.

Step 6. Follow procedure outlined in 6a or 6b depending on whether the joist capacity is controlled by flexure or by shear.

Step 6a. Joist design controlled by flexure.

a. Find ductility ratio $\mu = X_m/X_E$ from Figure 6-7 of TM 5-1300, using the values of $T/T_N$ and $B/r_u$.

b. Check if the ductility ratio and maximum end rotation meet the criteria requirements outlined in Section 2.3.3.

If the above requirements are not satisfied, select another dynamic load factor and repeat Steps 2 to 5.

c. Check if the top chord meets the requirements for a beam-column (Section 4.2).

Step 6b. Joist design controlled by shear.

a. Find ductility ratio $\mu = X_m/X_E$ from Figure 6-7 of TM 5-1300, using the values of $T/T_N$ and $B/r_u$.

b. If $\mu \leq 1.0$, design is O.K.

If $\mu > 1.0$, assume a higher dynamic load factor and repeat Steps 2 to 5. Continue until $\mu \leq 1.0$. Check end rotation, $\theta$, against design criteria.

c. Since the capacity is controlled by maximum end reaction, it will generally not be necessary to check the top chord as a beam-column. However, when such a check is warranted, the procedure in Step 6a.c can be followed.

Step 7. Check the bottom chord for rebound.

a. Determine the required resistance, r, for elastic behavior in rebound.

b. Compute the bending moment, M, and find the axial forces in top and bottom chords using

$$P = M/d$$

where d is taken as the distance between the centroids of the top and bottom chord sections.

c. Determine the ultimate axial load capacity of the bottom chord considering the actual slenderness ratio of its elements.

$$P_u = 1.7AF_a$$

where $F_a$ is defined in Section 4.2.

The value of $F_a$ can be obtained by using either Equation 4.3 or the tables in the AISC Specification which give allowable stresses for compression members. When using these tables, the yield stress should be taken equal to $F_{dy}$.

d. Check if $P_u > P$.

Determine bracing requirements.

Example 5(a): Design of an open-web steel joist

Required: Design a reusable simply-supported open-web steel joist whose capacity is controlled by flexure.

Solution:

Step 1. Given:

a. Pressure-time loading [Figure 7.4(a)]

b. Clear span $= 50'\text{-}0''$
Spacing of joists $= 7'\text{-}0''$
Weight of decking = 4 psf

c. Structural steel properties

Chords $F_y = 50{,}000$ psi
Web $F_y = 36{,}000$ psi

Dynamic increase factor

$C = 1.10$ (Table 2.1)

Dynamic yield stress, $F_{dy} = cF_y$

Chords $F_{dy} = 55{,}000$ psi
Web $F_{dy} = 39{,}600$ psi

d. Design criteria (Section 2.3.3)

Maximum ductility ratio: $\mu_{max} = 2.0$

Maximum end rotation: $\theta_{max} = 1^{o}$

Step 2. Selection of joist size

a. Assume a dynamic load factor. For a preliminary design of a reusable element, a DLF = 1.0 is generally recommended. However, since the span is quite long in this case, a DLF of 0.70 is selected.

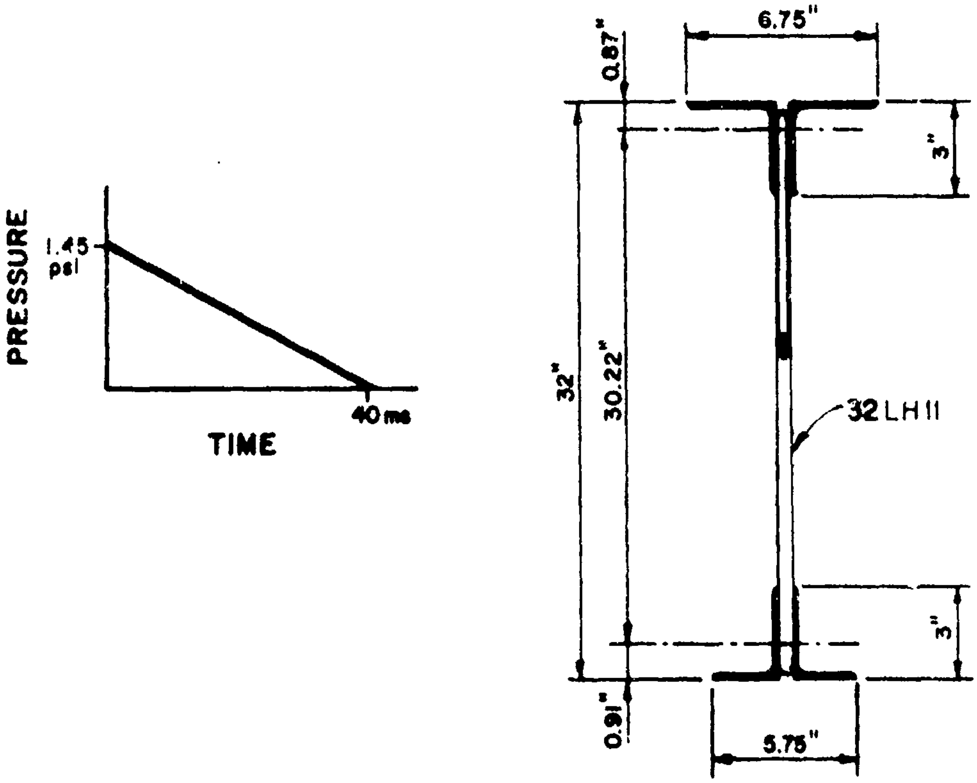
PRESSURE
1.45 psi
TIME
40 ms
6.75"
0.87"
3"
32"
30.22"
32 LH 11
3"
0.91"
5.75"

Figure 7.4(a) Joist cross-section and loading, Example 5(a).

b. Equivalent static load on joist:

$$w_1 = 0.70 \times 1.45 \times 144 \times 7.0$$

$$= 1{,}023 \text{ lb/ft}$$

c. Service live load on joist:

$$w_2 = w_1/1.87$$

$$= 1.023/1.87 = 547 \text{ lb/ft}$$

d. Using the "Standard Specification and Load Tables" of the Steel Joist Institute, for a span of 50'-0", try 32LH11. Joist tables show that capacity is controlled by flexure.

Total load-carrying capacity (including dead load = 602 lb/ft)

Approximate weight of joist and decking

$$= 28 + (4 \times 7) = 56 \text{ lb/ft}$$

Total load-carrying capacity (excluding dead load = 602 - 56 = 546 lb/ft)

The following section properties refer to the selected joist 32LH11 [Fig 7.4(a)]:

Top Chord:

Two 3 x 3 x 5/16 angles
$A = 3.56 \text{ in}^2$
$r_x = 0.92$ in.
$r_y = 1.54$ in.
$I_x = 3.02 \text{ in}^4$

Bottom Chord:

Two 3 x 2-1/2 x 1/4 angles
$A = 2.62 \text{ in}^2$
$r_x = 0.945$ in.
$r_y = 1.28$ in.
$I_x = 2.35 \text{ in}^4$

$I_{xx}$ for joist = $1{,}383.0 \text{ in}^4$

Panel length = 51 inches

**Step 3.** Resistance per unit length

$r_u = 1.87 \times 546 = 1{,}021$ lb/ft

**Step 4.** $K_E = 384\ EI/5L^3 = \dfrac{384 \times 29 \times 10^6 \times 1{,}383}{5(12 \times 50)^3}$ (Table 3.1)

$= 14{,}260$ lb/in

$X_E = r_u L/K_E = \dfrac{1{,}021 \times 50}{14{,}260} = 3.58$ inches

**Step 5.** Total mass of joist plus decking

$M = \dfrac{56 \times 50 \times 10^6}{386} = 7.25 \times 10^6$ lb-ms$^2$/in

Total effective mass $M_e = K_{LM}M$

$K_{LM} = 0.5(0.79 + 0.66) = 0.725$ (Table 6.1, TM 5-1300)

$M_e = 0.725(7.25 \times 10^6)$

$= 5.25 \times 10^6$ lb-ms$^2$/in

Natural period $T_N = 2\pi\sqrt{M_e/K_E}$

$= 2\pi\sqrt{5{,}250{,}000/14{,}260}$

$= 120.3$ ms

Behavior controlled by flexure. Use Step 6a.

**Step 6a.**

a. $T/T_N = 40/120.3 = 0.332$

$B/r_u = \dfrac{1.45 \times 144 \times 7}{1{,}021} = 1.43$

From Figure 6-7 of TM 5-1300,

$\mu = X_m/X_E = 1.42 < 2.0$ O.K.

b. $X_m = 1.42 \times 3.58 = 5.08$ inches

$\tan\theta = X_m/(L/2) = 5.08/(25 \times 12)$

$= 0.017 < 0.018$

$\theta \approx 1^\circ$ O.K.

c. Check top chord as a beam column.

Maximum moment at mid-span

$$M = r_u L^2/8 = \frac{1,021 \times (50)^2 \times 12}{8 \times 1,000}$$

$= 3,826$ in-kips

Maximum axial load in chords

$P = M/d$

$d$ = distance between centroids of top and bottom chords [see Figure 7.4(b)].

$= 30.22$ inches

$P = 3,826/30.22 = 126.6$ kips

$l$ = panel length = 51 inches

Slenderness ratio, $l/r_x = 51/0.92$

$= 55.4 < C_c$

where $C_c = \sqrt{2\pi^2 E/F_{dy}} = 102$ (Equation 4.1)

$F_a = 25.34$ ksi (Table 1-55, AISC Specification)

$P_u = 1.7AF_a$ (Equation 4.2)

$= 1.7 \times 3.56 \times 25.34 = 153.3$ kips

Considering the first panel as a fixed, simply-supported beam, the maximum moment in the panel is:

$$M = r_u L^2/12 = \frac{1{,}021(51)^2}{12 \times 12 \times 1000}$$

$$= 18.45 \text{ in-kips}$$

The effective slenderness ratio of the top chord in the first panel:

$$Kl_b/r_x = (1.0 \times 51)/0.92 = 55.4$$

$$F'_{ex} = \frac{12\pi^2 E}{23(Kl_b/r_x)^2} = \frac{12\pi^2 \times 29{,}000}{23(55.4)^2}$$

$$= 48.7 \text{ ksi}$$

$$P_{ex} = (23/12)AF'_{ex} = 23/12 \times 3.56 \times 48.7$$

$$= 333 \text{ kips}$$

$$(1 - P/P_{ex}) = (1.0 - 126.6/333) = 0.62$$

To determine $M_m$, the plastic moment $M_p$ is needed and the value of $Z_x$ has to be computed.

The neutral axis for a fully plastic section is located at a distance $\bar{x}$ from the flange.

$$3\bar{x} = (3 - 5/16)5/16 + 3(5/16 - \bar{x})$$

$$= (43)5/(16 \times 16) + 15/16 - 3\bar{x}$$

$$\bar{x} = 455/(6 \times 256) = 0.296 \text{ inch}$$

The plastic section modulus, $Z_x$, is found to be:

$$Z_x = 2[\frac{(0.296)^2}{2} \times 3$$

$$+ (3.0 - 0.3125)\frac{(0.3125 - 0.296)^2}{2}$$

$$+ \frac{(3 - 0.296)^2}{2} \times 0.3125]$$

$$= 0.263 + 0.007 + 2.285 = 2.555 \text{ in}^3$$

$M_{px} = F_{dy}Z_x = 55 \times 2.555 = 140.5$ in-kips
(Section 4.2)

$$M_{mx} = [1.07 - \frac{(l/r_y)\sqrt{F_{dy}}}{3,160}]M_{px} \leq M_{px}$$
(Equation 4.7)

where $r_y$ is least radius of gyration = 0.92

$= [1.07 - (55.4/427)]140.5$

$= (1.07 - 0.13)140.5 = 132.0$ in-kips

$C_m = 0.85$ (Section 1.6.1, AISC)

$P/P_u + C_mM/[(1 - P/P_{ex})M_{mx}] \leq 1.0$
(Equation 4.4)

$$\frac{126.6}{153.3} + \frac{0.85(18.45)}{0.62(132.0)} \leq 1.0$$

$= 0.825 + 0.192 = 1.017 \approx 1.0$ O.K.

Step 7. Check bottom chord for rebound.

a. Calculate required resistance in rebound.

$T/T_N = 0.332$

From Figure 3.8, 100% rebound

$\bar{r}/r_u = 1.0$

$\bar{r} = r_u = 1,021$ lb/ft.

b. Moment and axial forces in rebound

$M = (\bar{r} \times L^2)/8 = 3,826$ in-kips

Maximum axial force in bottom chord

$P = M/d = 126.6$ kips (compression)

c. Ultimate axial load capacity

Stability in vertical direction
(about x-axis)

$l = 51$ inches $\quad r_x = 0.945$

$l/r = 51/0.945 = 54.0 < C_c$

where $C_c = \sqrt{(2\pi^2 E)/F_{dy}} = 102$

$F_a = 25.61$ ksi (Table 1-55, AISC Specification)

$P_u = 1.7AF_a = 1.7 \times 2.62 \times 25.61$

$= 114.0$ kips

d. Check bracing requirements.

$(P_u = 114.0) < (P_u = 126.6)$ N.G.

Adding a vertical member (L 2x2x3/16) between panel joints of bottom chord

New slenderness ratio $= \frac{(51.0)}{2}/0.945$

$= 27.0$

In this case, $F_a = 30.09$ ksi

$P_u = 1.7 \times 2.62 \times 30.09$

$= 134.0$ kips $> 126.6$ kips O.K.

(This additional bracing is needed in mid-span but may be spared at the joist ends.)

Stability in the lateral direction (about y-axis)

$P_u = 126.6$ kips

$r_y = 1.28$ inches, $\quad A = 2.62$ in$^2$

$F_a = P_u/(1.7A) = 126.6/(1.7 \times 2.62)$

$= 28.4$ ksi

For a given $F_a = 28.4$, the corresponding slenderness ratio:

$l/r = 38$ (Table 1-55, AISC Specification)

Therefore, maximum unbraced length in mid-span:

$l_b = 38 \times 1.28 = 48.6$ inches

Use lateral bracing at panel points, i.e., 51 inches at mid-span. The unbraced length may be increased at joist ends, but not greater than specified for bridging requirements in the joist specification.

Example 5(b): Analysis of existing open-web steel joist.

Required: Analyze a simply-supported, reusable open-web steel joist whose capacity is controlled by shear.

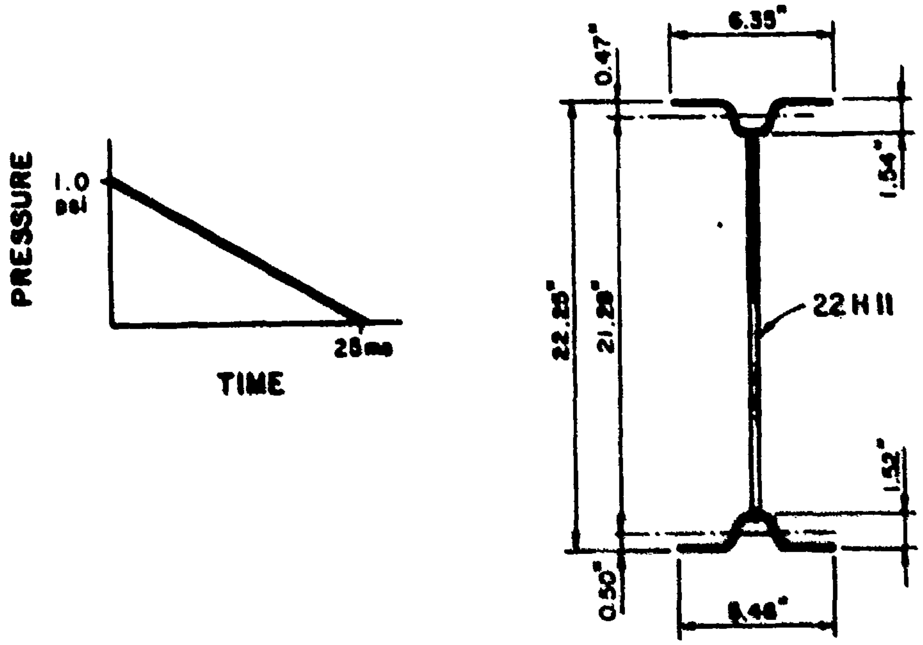
PRESSURE
1.0 psi
28ms
TIME
6.35"
0.47"
1.54"
22.25"
21.28"
22 H 11
1.52"
0.50"
6.46"

Figure 7.4(b) Joist cross-section and loading, Example 5(b).

Solution:

Step 1. Given:

a. Pressure-time loading [Figure 7.4(b)]

Joist 22H11

b. Clear span = 32'-0"

Spacing of joists = 6'-0"

Weight of decking = 4 psf

c. Properties of structural steel:

Chords $F_y = 50{,}000$ psi

Web $F_y = 36{,}000$ psi

Dynamic increase factor, c = 1.10 (Table 2.1)

Dynamic yield strength, $F_{dy} = cF_y$

Chords $F_{dy} = 55{,}000$ psi

Web $F_{dy} = 39{,}600$ psi

d. Design criteria (Section 2.3.3)

$\mu_{max} = 1.0$

$\theta_{max} = 1^o$

Step 2.

a. Assume the DLF = 1.0

b. Overpressure load on joist

$w_1 = 1.0 \times 1.0 \times 144 \times 6 = 864$ lb/ft

c. Equivalent service load

$w_2 = w_1/1.87 = 864/1.87 = 462$ lb/ft

d. From the "Standard Specifications and Load Tables" of the Steel Joist Institute:

Total load-carrying capacity (including dead load) = 506 lb/ft

Approximate weight of joist plus decking =

17 + (6 x 4) = 41 lb/ft

Total load-carrying capacity (excluding dead load) =

506 - 41 = 465 > 462      O.K.

From the steel joist catalog, the following are the section properties of Joist 22H11 [Figure 7.4(b)]:

Panel length = 24 inches

Top Chord:

$A = 1.935 \text{ in}^2$
$I_x = 0.455 \text{ in}^4$
$r_x = 0.485 \text{ in.}$
$r_y = 1.701 \text{ in.}$

Bottom Chord:

$A = 1.575 \text{ in}^2$
$I_x = 0.388 \text{ in}^4$
$r_x = 0.497 \text{ in.}$
$r_y = 1.469 \text{ in.}$

$I_{xx}$ for joist = $396.0 \text{ in}^4$

Step 3. Resistance per unit length

$r_u = 1.87 \times 465 = 870$ lb/ft

Step 4. $$K_E = \frac{384EI}{5L^3} = \frac{384 \times 29 \times 10^6 \times 396}{5 \times (12 \times 32)^3}$$

$= 15,580$ lb/in      (Table 3.1)

$$X_E = r_u L/K_E = \frac{870 \times 32}{15,580} = 1.79 \text{ inches}$$

**Step 5.** Mass of joist plus decking

$$M = \frac{41 \times 32 \times 10^6}{386} = 3.4 \times 10^6 \text{ lb-ms}^2/\text{in}$$

Effective mass $M_e = K_{LM} M$ (Table 6-1, TM 5-1300)

$$= 0.78 \times 3.4 \times 10^6 = 2.65 \times 10^6 \text{ lb-ms}^2/\text{in}$$

Natural period $T_N = 2\pi\sqrt{M_e/K_E}$

$$= 2\pi\sqrt{2,650,000/15,580} = 81.8 \text{ ms}$$

Behavior controlled by shear. Use Step 6b of the procedure.

**Step 6b.**

a. $T/T_N = 25/81.8 = 0.305$

$$B/r_u = \frac{6 \times 144 \times 1.0}{870} = 0.993 \approx 1.0$$

From Fig. 6-7 of TM 5-1300,

b. $\mu = X_m/X_E < 1.0$; elastic, O.K.

$\tan\theta < 1.79/(L/2)$

$= 1.79/(16.0 \times 12) = 0.0093$

$\theta = 0.53^\circ < 1^\circ$ O.K.

c. Check of top chord as a beam-column is not necessary.

**Step 7.** Check bottom chord in rebound.

a. For $\mu = 1$ and $T/T_N = 0.305$,

rebound = 100% (Figure 3.9)

$r = r_u$

b. Determine axial load in bottom chord, $P = M/d$.

For an elastic response, $\mu < 1.0$, where

$T/T_N = 0.305$, the DLF $= 0.87$

(Figure 3.3)

Equivalent static load, w

$w = \text{DLF} \times b \times p$

$= 0.87 \times 12 \times 12 \times 6 \times 1.0 = 751$ lb/ft

Maximum moment in rebound, $M = wL^2/8$

$M = 751 \times (32)^2/8 \times 12 = 1{,}155{,}000$ in-lb

$P = M/d = 1{,}155.000/21.28$

$= 54{,}300$ lb $= 54.3$ kips

c. Check bracing requirements.

(1) Vertical bracing of bottom chord:

Panel length = 24 inches

$r_x = 0.497$, $r_y = 1.469$, $A = 1.575$ in$^2$

$l/r_x = 24/0.497 = 48.3$

Allowable $P = 1.7 \times 1.575 \times 26.67$

$= 71.4$ kips $> 54.3$ kips

(Table 1-55,
AISC Specification)

No extra bracing required.

(2) Lateral bracing of bottom chord:

$P = 54.3$ kips, $A = 1.575$ in$^2$

$F_a = P/1.7A = 54.3/(1.7 \times 1.575)$

$= 20.3$ ksi

For $F_{dy} = 55$ ksi and $F_a = 20.3$ ksi

$l/r = 79$

$l = 79 \times 1.469 = 116$ inches
(Table 1-55,
AISC Specification)

Therefore, use lateral bracing at every 4th panel point close to mid-span. The unbraced length may be increased at joist ends, but not greater than specified for bridging requirements in the joist specification.

## 7.6 Design of Single-Story Rigid Frames for Pressure-Time Loading

Problem 6: Design a single-story, multi-bay rigid frame subjected to a pressure-time loading.

Procedure:

Step 1. Establish the ratio $\alpha$ between the design values of the horizontal and vertical blast loads.

Step 2. Using the recommended dynamic load factors presented in Table 5.2, Section 5.2.1, establish the magnitude of the equivalent static load w for:

(a) local mechanisms of the roof and blastward column, and

(b) panel or combined mechanisms for the frame as a whole.

Step 3. Using the general expressions for the possible collapse mechanisms from Table 5.1 and the loads from Step 2, assume values of the moment capacity ratios C and $C_1$ and proceed to establish the required design plastic moment $M_p$ considering all possible mechanisms. In order to obtain a reasonably economical design, it is desirable to select C and $C_1$ so that the least resistance (or the required value of $M_p$) corresponds to a combined mechanism. This will normally require several trials with assumed values of C and $C_1$.

Step 4. Calculate the axial loads and shears in all members using the approximate method of Section 5.2.4.

Step 5. Design each member as a beam-column using the ultimate strength design criteria of Chapter 4. A numerical example is presented in Section 7.4.

Step 6. Using the moments of inertia from Step 5, calculate the sidesway natural period using Table 5.3 and Equations 5.1 and 5.2. Enter Figure 6-7 of TM 5-1300 with the ratios of $T/T_N$ and $B/R_u$ and establish the ductility ratio $\mu$. In this case, $B/R_u$ is the reciprocal of the panel or sidesway mechanism dynamic load factor used in the trial design. Multiply the ductility ratio by the elastic deflection given by Equation 5.4 and establish the peak deflection $X_m$ from Equation 5.5. Compare the $X_m/H$ with the criteria of Chapter 2.

Step 7. Repeat the procedure of Step 6 for the local mechanisms of the roof and blastward column. The stiffness and natural period may be obtained from Tables 3.1 and 3.3, respectively. The resistance of the roof girder and the blastward column may be obtained from Table 5.1 using the values of $M_p$ and $CM_p$ determined in Step 3. Compare the ductility ratio and rotation with the criteria of Chapter 2.

Step 8. a. If the deflection criteria for both sidesway and beam mechanisms are satisfied, then the member sizes from Step 5 constitute the results of this preliminary design. These members would then be used in a more rigorous dynamic frame analysis such as the nonlinear dynamic computer program DYNFA.

b. If the deflection criterion for a sidesway mechanism is exceeded, then the resistance of all or most of the members should be increased.

c. If the deflection criterion for a beam mechanism of the front wall or roof girder is exceeded, then the resistance of the member in question should be increased. The member sizes to be used in a final analysis should be the greater of those determined from Steps 8b and 8c.

**Example 6:** Design of a rigid frame for pressure-time loading.

**Required:** Design a four-bay, single-story, reusable, pinned-base rigid frame subjected to a pressure-time loading in its plane.

Given:

a. Pressure-time loading (Figure 7.5)

b. Design criteria for a reusable structure. For a frame, $\delta/H = 1/50$. The limits on $\theta_{max}$ for a frame member are summarized in Section 2.3.3.

c. Structural configuration (Figure 7.5)

d. A36 steel

e. Roof purlins spanning perpendicular to frame

f. Frame spacing, $b = 17$ ft

g. Uniform dead load of deck, excluding frame

Step 1. Determine $\alpha$: (Section 5.2.1)

$$b_h = b_v = 17 \text{ ft}$$

$$q_h = 5.8 \times 17 \times 12 = 1{,}183 \text{ lbs/in}$$

$$q_v = 2.5 \times 17 \times 12 = 510 \text{ lbs/in}$$

$$\alpha = q_h/q_v = 2.32$$

Step 2. Establish equivalent static loads. (Table 5.2)

a. Local beam mechanism, $w = DLF \times q_v$

$$w = \frac{1.0 \times 510 \times 12}{1{,}000} = 6.12 \text{ k/ft.}$$

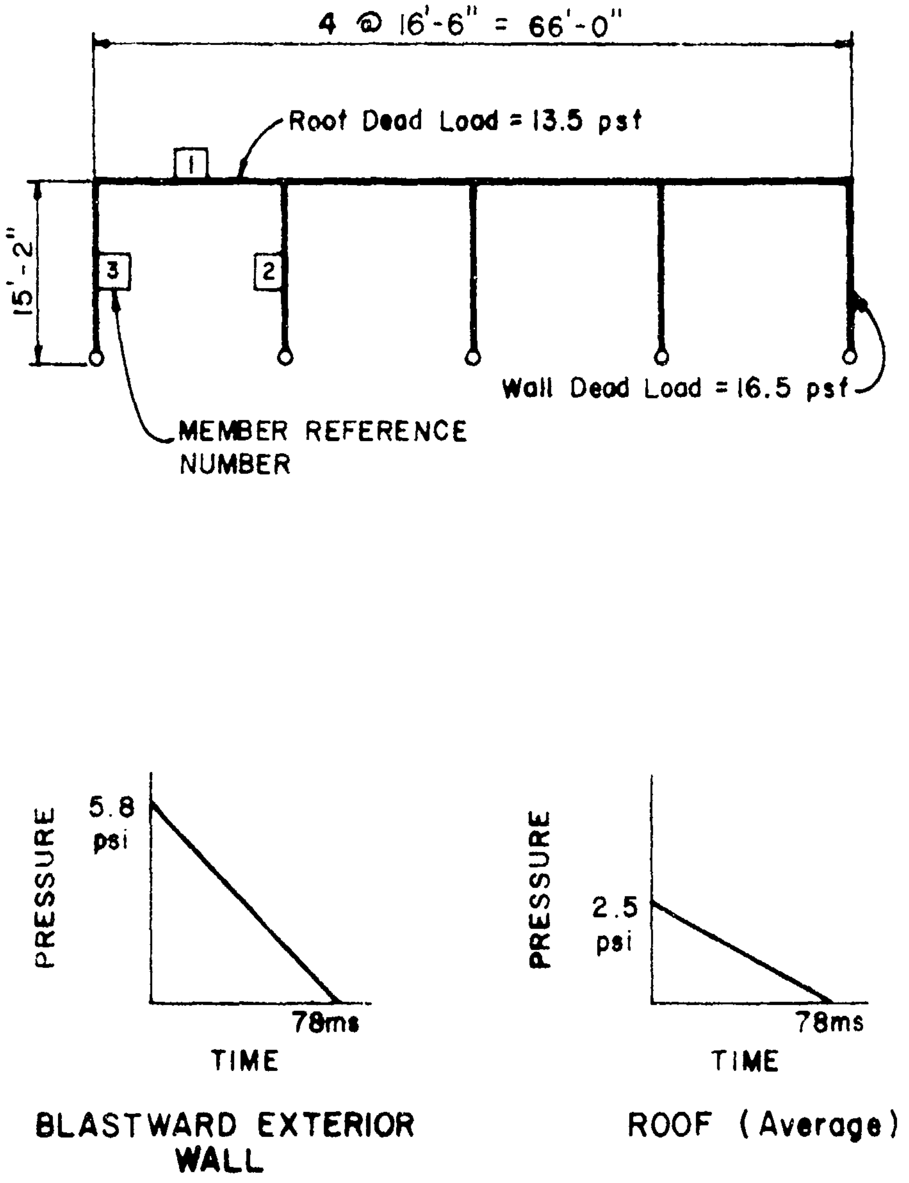
4 @ 16'-6" = 66'-0"
Roof Dead Load = 13.5 psf
1
15'-2"
3
2
Wall Dead Load = 16.5 psf
MEMBER REFERENCE NUMBER
PRESSURE
5.8 psi
78ms
TIME
PRESSURE
2.5 psi
78ms
TIME
BLASTWARD EXTERIOR WALL
ROOF (Average)

Figure 7.5 Preliminary design of four-bay, single-story rigid frame, Example 6.

b. Panel or combined mechanism, $w = \text{DLF} \times q_v$

$$w = \frac{0.5 \times 510 \times 12}{1{,}000} = 3.06 \text{ k/ft}$$

Step 3. The required plastic moment capacities for the frame members are determined from Table 2.1 based upon rational assumptions for the moment capacity ratios $C_1$ and $C$. In general, the recommended starting values are $C_1$ equal to 2 and $C$ greater than 2.

From Table 5.1, for $n = 4$, $\alpha = 2.32$,

$H = 15.167$ ft, $L = 16.5$ ft

and pinned bases, values of $C_1$ and $C$ were substituted and after a few trials, the following solution is obtained:

$M_p = 104$ kip-ft, $C_1 = 2.0$ and $C = 3.5$.

The various collapse mechanisms and the associated values of $M_p$ are listed below:

| Collapse Mechanism | w (k/ft) | $M_p$ (k-ft) |
|---|---|---|
| 1 | 6.12 | 104 |
| 2 | 6.12 | 102 |
| 3a, 3b | 3.06 | 102 |
| 4 | 3.06 | 103 |
| 5a, 5b | 3.06 | 88 |
| 6 | 3.06 | 93 |

The plastic design moments for the frame members are established as follows:

Girder, $M_p = 104$ k-ft

Interior column, $C_1M_p = 208$ k-ft

Exterior column, $CM_p = 364$ k-ft

Step 4. a. Axial loads and shears due to horizontal blast pressure,

$$w = 3.06 \text{ k/ft}$$

From Figure 5.1, $R = \alpha wH$

$$= 2.32 \times 3.06 \times 15.167$$

$$= 108 \text{ kips}$$

(1) Member 1, axial load

$$P_1 = R/2 = 54 \text{ kips}$$

(2) Member 2, shear force

$$V_2 = R/2(4) = 108/8 = 13.5 \text{ kips}$$

(3) Member 3, shear force

$$V_3 = R/2 = 54 \text{ kips}$$

b. Axial loads and shears due to vertical blast pressure,

$$w = 6.12 \text{ k/ft}$$

(1) Member 1, shear force

$$V_1 = w \times L/2 = 6.12 \times 16.5/2$$

$$= 50.4 \text{ kips}$$

(2) Member 2, axial load

$$P_2 = w \times L = 6.12 \times 16.5$$

$$= 101.0 \text{ kips}$$

(3) Member 3, axial load

$$P_3 = w \times L/2 = 50.4 \text{ kips}$$

Note: The dead loads are small compared to the blast loads and are neglected in this step.

**Step 5.** The members are designed using the criteria of Chapter 4 with the following results:

| Member | $M_p$ (k-ft) | P (k) | V (k) | USE | $I_x$ ($in^4$) |
|---|---|---|---|---|---|
| 1 | 104 | 54.0 | 50.4 | W12x36 | 281 |
| 2 | 208 | 101.0 | 13.6 | W14x61 | 641 |
| 3 | 364 | 50.4 | 54.0 | W14x78 | 851 |

**Step 6.** Determine the frame stiffness and sway deflection.

$$I_{ca} = \frac{(3 \times 641) + (2 \times 851)}{5} = 725 \text{ in}^4$$

(Table 5.3)

$I_g = 281 \text{ in}^4$

$\beta = 0$

$$D = \frac{I_g/L}{0.75 I_{ca}/H} = \frac{281/16.5}{(0.75)(725/15.167)} = 0.475$$

(Table 5.3)

Using linear interpolation to get $C_2$

$C_2 = 4.49$

$$K = \frac{EI_{ca}C_2}{H^3}[1 + (0.7 - 0.1\beta)(n-1)]$$

(Table 5.3)

$$= \frac{(30)(10^3)(725)(4.49)[1 + 0.7(3)]}{(15.167 \times 12)^3}$$

$= 50.2$ k/in (Equation 5.2)

$K_L = 0.55(1 - 0.25\beta) = 0.55$

Calculate dead weight, W:

$W = b[(4Lw_{dr}) + (2/3)(Hw_{dw})]$

$+ (36 \times 66) + 1/3(15.167)[(2 \times 61) + (3 \times 78)]$

$= 20{,}740$ lbs

$m_e = W/g = 20{,}740/32.2 = 644$ lb-sec$^2$/ft

$= 644 \times 10^6$ lb-ms$^2$/ft

$T_N = 2\pi\sqrt{m_e/KK_L}$

$= 2\pi\sqrt{(644 \times 10^6)/(50.2 \times 12 \times 10^3 \times 0.55)}$

(Equation 5.2)

$= 277$ ms

$T/T_N = 78/277 = 0.282$

$B/R_u = 2.0$

$\mu = X_m/X_E = 1.90$ (Figure 6-7, TM 5-1300)

$X_E = R_u/K = \alpha wH/K$ (Equations 5.4 and 5.5)

$$= \frac{2.32 \times 3.06 \times 15.167}{50.2} = 2.14 \text{ inches}$$

$X_m = \delta = 1.90 \times 2.14 = 4.06$ inches (Equation 5.6)

$\delta/H = 4.06/(15.167)(12) = 0.0223 \simeq 1/50$

Step 7. Check deflection of possible local mechanisms.

a. Roof girder mechanism (investigate W12 x 36 from Step 5)

$T_N = 0.28L^2\sqrt{(w/g)/EI}$ (Table 3.2)

$w = (13.5 \times 17) + 36 = 265$ lb/ft

$I_g = 281$ in$^4$

$EI_g = 30 \times 10^6 \times 281/144$

$= 58.4 \times 10^6$ lb-ft$^2$

$L = 16.5$ ft

$T_N = (0.28)(16.5)^2\sqrt{265(10^6)/(32.2)(58.4)(10^6)}$

$= 28.6$ ms

$T/T_N = 78/28.6 = 2.73$

$R_u = 16M_p/L = (16 \times 104)/16.5 = 101$ kips
(Table 5.1)

$B = pbL$

$= (2.5)(17)(144)(16.5)/1,000 = 101$ kips

$B/R_u = 101/101 = 1.0$

$\mu = X_m/X_E = 3.50 > 3$ N.G.
(Figure 6-7, TM 5-1300)

Check end rotation of girder.

$$K = 307EI/L^3 = \frac{(307)(30)(10^3)(281)}{(16.5 \times 12)^3}$$

$= 332$ k/in (Table 3.1)

$X_E = R_u/K = 101/332 = 0.304$ inch

$X_m = 3.50 \times 0.304 = 1.06$ inches

$X_m/(L/2) = 1.06/(8.25)(12) = 0.0107$

$= \tan \theta, \quad \theta = 0.6^\circ < 1^\circ$ O.K.

b. Exterior column mechanism (investigate W14 x 78 from Step 5).

$T_N = 0.42L^2\sqrt{(w/g)/EI}$ (Table 3.2)

$w = (16.5 \times 17) + 78 = 358$ lb/ft

$EI = (30)(10^6)(851/144) = 177 \times 10^6$ lb-ft$^2$

$L = 15.167$ ft

$T_N = (0.42)(15.167)^2\sqrt{358(10^6)/(32.2)(177)(10^6)}$

$= 24.2$ ms

$$T/T_N = 78/24.2 = 3.22$$

$$R_u = \frac{4M_p(2C + 1)}{H} = \frac{4(104)[(2 \times 3.5) + 1]}{15.167}$$

$= 220$ kips (Table 5.1)

$$B = (2.32)(6.12)(15.167) = 215 \text{ kips}$$

$$B/R_u = 215/220 = 0.98$$

$\mu = X_m/X_E = 3.60 > 3$ N.G. (Figure 6-7, TM 5-1300)

Check end rotation of columns.

$$K = \frac{160EI}{L^3} = \frac{(160)(30)(10^3)(851)}{(15.167 \times 12)^3}$$

$= 676$ k/in (Table 3.1)

$$X_E = R_u/K = 220/676 = 0.325 \text{ inch}$$

$$X_m = 3.60 \times 0.325 = 1.17 \text{ inches}$$

$$X_m/(L/2) = 1.17/(7.58)(12) = 0.0129 = \tan\theta$$

$\theta = 0.74^\circ < 1^\circ$ O.K.

Step 8.

a. The deflections of the local mechanisms exceed the criteria. The sidesway deflection is acceptable.

b. Roof girder

$\mu = 3.50$ from Step 7; increase trial size from W12 x 36 to W12 x 40.

c. Front wall

$\mu = 3.60$ from Step 7; increase trial size from W14 x 78 to W14 x 84.

Summary: The member sizes to be used in a computer analysis are as follows:

| Member | Size |
|---|---|
| 1 | W12 x 40 |
| 2 | W14 x 61 |
| 3 | W14 x 84 |

## 7.7 Design of Doors for Pressure-Time Loading.

Problem 7: Design a steel-plate blast door subjected to a pressure-time loading.

Procedure:

Step 1. Establish the design parameters

a. Pressure-time load

b. Design criteria, $\mu_{max}$ and $\theta_{max}$ for a reusable or non-reusable structure (Section 2.3.3)

c. Structural configuration of the door including geometry and support conditions

d. Properties of steel used:

Minimum yield strength, $F_y$, for door components

Dynamic increase factor, c (Table 2.1)

Step 2. Select the thickness of the plate.

Step 3. Calculate the elastic section modulus, S, and the plastic section modulus, Z, of the plate.

Step 4. Calculate the design plastic moment, $M_p$, of the plate (Equation 3.2).

Step 5. Compute the ultimate dynamic shear, $V_p$ (Equation 3.11).

Step 6. Calculate maximum support shear, V, using a dynamic load factor of 1.0 and determine $V/V_p$.

If $V/V_p$ is less than 0.67, use the plastic design moment as computed in Step 4 (Section 3.4.3).

If $V/V_p$ is greater than 0.67, use Equation 3.18 to calculate the effective $M_p$.

Step 7. Calculate the ultimate unit resistance of the section (Table 5-5 of TM 5-1300), using the equivalent plastic moment as obtained in Step 4 and a dynamic load factor of 1.0.

Step 8. Determine the moment of inertia of the plate section.

Step 9. Compute the equivalent elastic unit stiffness, $K_E$, of the plate section (Table 3.1).

Step 10. Calculate the equivalent elastic deflection, $X_E$, of the plate as given by $X_E = r_u/K_E$.

Step 11. Determine the load-mass factor $K_{LM}$ and compute the effective unit mass, $m_e$.

Step 12. Compute the natural period of vibration, $T_N$, (Equation 3.10).

Step 13. Determine the door response using the values of $B/r_u$ and $T/T_N$ with Figure 6-7 of TM 5-1300 or Figure 3.3 to determine the values of $X_m/X_E$ and $\theta$.

Compare with design criteria of Step 1. If these requirements are not satisfied, select another thickness and repeat Steps 2 to 13.

Step 14. Design supporting flexural element considering composite action with the plate.

Step 15. Calculate elastic and plastic section moduli of the combined section.

Step 16. Follow the design procedure for a flexural element as described in Section 7.1.

**Example 7:** Design of a blast door for pressure-time loading.

**Required:** Design a reusable double-leaf door (6'-0" x 8'-0") for the given pressure-time loading.

Step 1. Given:

a. Pressure-time loading (Figure 7.6)

b. Design criteria: maximum ductility ratio, $\mu_{max} = 5$, maximum end rotation, $\theta_{max} = 2^{o}$, whichever governs (Section 2.3.3)

c. Structural configuration (Figure 7.6)

NOTE: This type of door configuration is suitable for low-pressure range applications (5 to 15 psi).

d. Steel used: A36

Yield strength, $F_y = 36$ ksi (Section 2.2.1)

Dynamic increase factor, $c = 1.1$ (Table 2.1)

Hence, the dynamic yield strength,

$F_{dy} = 1.1 \times 36 = 39.6$ ksi (Equation 2.1)

and the dynamic yield stress in shear,

$$F_{dv} = 0.55F_{dy}$$

$$= 0.55 \times 39.6 = 21.78 \text{ ksi} \quad \text{(Equation 2.2)}$$

Step 2. Assume a plate thickness of 5/8 inch.

Step 3. Determine the elastic and plastic section moduli (per unit width).

$$S = \frac{bd^2}{6} = \frac{1 \times (5/8)^2}{6}$$

$$= 6.515 \times 10^{-2} \text{ in}^3/\text{in}$$

$$Z = \frac{bd^2}{4} = \frac{1 \times (5/8)^2}{4}$$

$$= 9.765 \times 10^{-2} \text{ in}^3/\text{in}$$

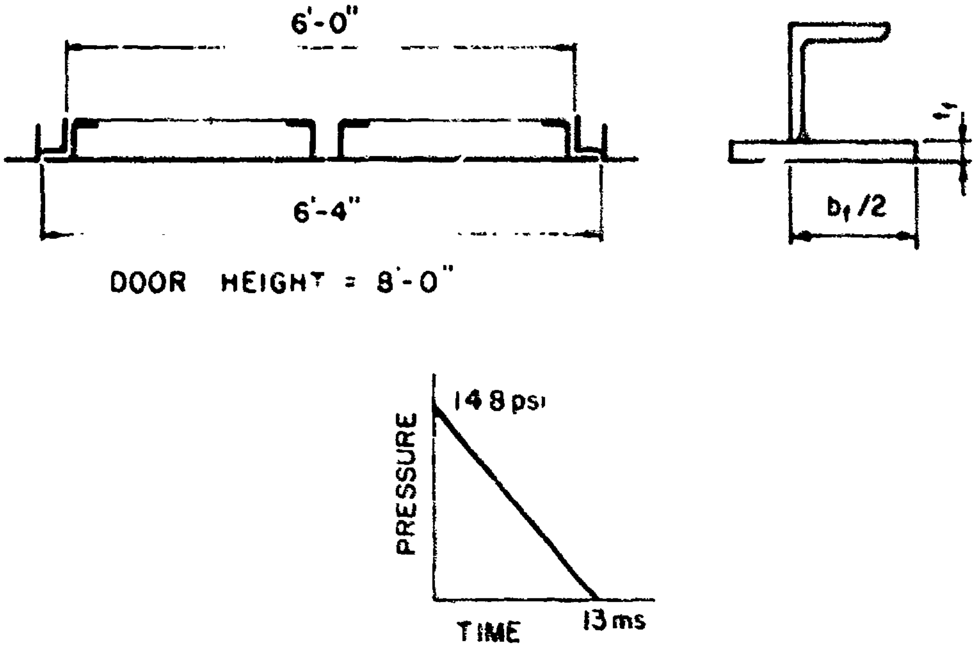
6'-0"
6'-4"
DOOR HEIGHT = 8'-0"
$b_f/2$
14.8 psi
PRESSURE
TIME
13 ms

Figure 7.6 Door configuration and loading, Example 7.

**Step 4.** **Calculate the design plastic moment, $M_p$.**

$$M_p = F_{dy}(S + Z)/2 \quad \text{(Equation 3.2)}$$

$$= 39.6[(6.515 \times 10^{-2})$$

$$+ (9.765 \times 10^{-2})]/2$$

$$= 39.6 \times 8.14 \times 10^{-2}$$

$$= 3.225 \text{ in-k/in}$$

**Step 5.** **Calculate the dynamic ultimate shear capacity, $V_p$, for a 1-inch width.**

$$V_p = F_{dv}A_v \quad \text{(Equation 3.11)}$$

$$= 21.78 \times 1 \times 5/8 = 13.61 \text{ kips/in}$$

Step 6. Evaluate the support shear and check the plate capacity. Assume DLF = 1.0.

$$V = DLF \times B \times L/2 = \frac{1.0 \times 4.8 \times 36 \times 1}{2}$$

$$= 266.4 \text{ lbs/in} = 0.2664 \text{ kip/in}$$

$$V/V_p = 0.2664/13.61 = 0.01958 < 0.67$$

(Section 3.4.3)

No reduction in equivalent plastic moment is necessary.

NOTE: When actual DLF is determined, reconsider Step 6.

Step 7. Calculate the ultimate unit resistance, $r_u$, (assuming the plate to be simply-supported at both ends).

$$r_u = 8M_p/L^2 \qquad \text{(Table 3.1)}$$

$$= \frac{8 \times 3.225 \times 10^3}{(36)^2} = 19.92 \text{ psi}$$

Step 8. Compute the moment of inertia, I, for a 1-inch width.

$$I = \frac{bd^3}{12} = \frac{1 \times (5/8)^3}{12} = 0.02035 \text{ in}^4/\text{in}$$

Step 9. Calculate the equivalent elastic stiffness, $K_E$.

$$K_E = 384EI/5bL^4 \qquad \text{(Table 3.1)}$$

$$= \frac{384 \times 29 \times 10^6 \times .02035}{5 \times 1 \times (36)^4}$$

$$= 27.0 \text{ psi/in}$$

Step 10. Determine the equivalent elastic deflection, $X_E$.

$$X_E = r_u/K_E = 19.92/27.0 = 0.738 \text{ inch}$$

Step 11. Calculate the effective mass of element.

a. $K_{LM}$ (average elastic and plastic)

$= (0.79 + 0.66)/2 = 0.725$

b. Unit mass of element, m

$$m = w/g = \frac{5/8 \times 1 \times 1 \times 490 \times 10^6}{1{,}728 \times 32.2 \times 12}$$

$= 458.0$ psi-ms$^2$/in

c. Effective unit mass of element, $m_e$

(Section 6.6,
TM 5-1300)

$m_e = K_{LM}m = 0.725 \times 458.0$

$= 332$ psi-ms$^2$/in

Step 12. Calculate the natural period of vibration, $T_N$.

$T_N = 2\pi\sqrt{332/27.0} = 22.05$ ms

Step 13. Determine the door response.

Peak overpressure $B = 14.8$ psi

Peak resistance $r_u = 19.92$ psi

Duration $T = 13.0$ ms

Natural period of vibration $T_N = 22.05$ ms

$B/r_u = 14.8/19.92 = 0.743$

$T/T_N = 13.0/22.05 = 0.59$

From Figure 6-7, TM 5-1300,

$X_m/X_E < 1$, so it satisfies the ductility ratio criterion

Since the response is elastic, determine the DLF from Figure 3.3.

DLF $= 1.3$ for $T/T_N = 0.59$

Hence, $X_m = \dfrac{1.3 \times 14.8 \times 0.738}{19.92} = 0.713$ inch

$\tan\theta = X_m/(L/2) = 0.713/(36/2) = 0.0396$

$\theta = 2.27^\circ > 2^\circ$ N.G.

Since the rotation criterion is not satisfied, change the thickness of the plate and repeat the procedure. Repeating these calculations, it can be shown that a 3/4-inch plate satisfies the requirements.

Step 14. Design of the supporting flexural element.

Assume an angle L4 x 3 x 1/2 and attached to the plate as shown in Figure 7.7.

Determine the effective width of plate which acts in conjunction with the angle

$b_f/2t_f \leq 8.5$ (Section 3.3.4)

where $b_f/2$ is the half width of the outstanding flange or overhang and $t_f$ is the thickness of the plate.

With $t_f = 3/4$ inch,

$b_f/2 \leq 8.5 \times 3/4$, i.e., 6.38 inches

Use an overhang of 6 inches.

Hence, the effective width = 6 + 2 = 8 inches.

The angle together with plate is shown in Figure 7.7.

Step 15. Calculate the elastic and plastic section moduli of the combined section.

Let $\bar{y}$ be the distance of c.g. of the combined section from the outside edge of the plate as shown in Figure 7.7, therefore

$$\bar{y} = \frac{(8 \times 3/4 \times 3/8) + (4 + 3/4 - 1.33) \times 3.25}{(8 \times 3/4) + 3.25}$$

= 1.445 inches

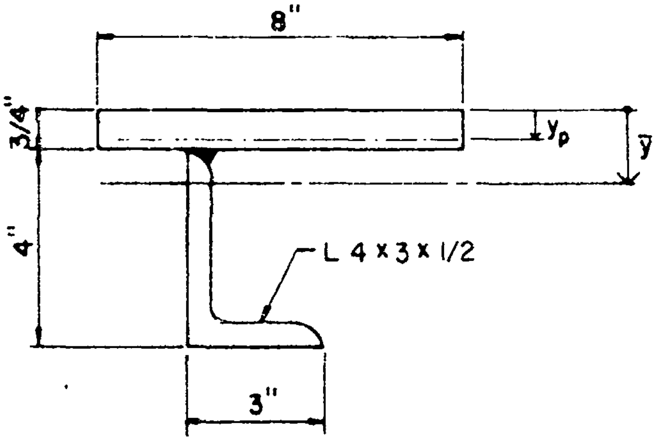

Figure 7.7 Detail of composite angle/plate supporting element, Example 7.

**Let $y_p$ be the distance to the N.A. of the combined section for full plasticity.**

$$y_p = \frac{1}{8 \times 2}\left[(8 \times 3/4) + 3.25\right] = 0.578 \text{ inch}$$

$$I = \frac{8 \times (3/4)^3}{12} + 8 \times 3/4 \times (1.445 - 3/8)^2$$

$$+ 5.05 + 3.25(4 + 3/4 - 1.33 - 1.445)^2$$

$$= 24.881 \text{ in}^4$$

Hence, $S_{min} = 24.881/(4.75 - 1.445)$

$$= 7.54 \text{ in}^3$$

$$Z = 8(0.578)^2/2 + 8(0.75 - 0.578)^2/2$$

$$+ 3.25(4.75 - 0.937 - 0.578)$$

$$= 11.97 \text{ in}^3$$

Step 16. **Calculate the design plastic moment $M_p$ of the supporting flexural element.**

$$M_p = 39.6(7.54 + 11.97)/2 \qquad \text{(Equation 3.2}$$

$$= 384.5 \text{ in-kips}$$

Calculate the ultimate dynamic shear capacity, $V_p$.

$V_p = F_{dv}A_w$ (Equation 3.11)

$= 21.78(4.0 - 1/2)1/2$

$= 38.1$ kips

Calculate support shear and check shear capacity.

L = 8'-0" = 96 inches

$V = (14.8 \times 36/2 \times 96)/2 = 12{,}790$ lbs

$= 12.79$ kips $< V_p$ O.K. (Section 3.3.3)

Calculate the ultimate unit resistance, $r_u$.

Assuming the angle to be simply supported at both ends:

$r_u = 8M_p/L^2$ (Table 3.1)

$= (8 \times 384.5 \times 1{,}000)/(96)^2$

$= 333.5$ lbs/in

Calculate the unit elastic stiffness, $K_E$.

$K_E = 384EI/5L^4$ (Table 3.1)

$$= \frac{384 \times 29 \times 10^6 \times 24.881}{5 \times (96)^4} = 652.5 \text{ lbs/in}^2$$

Determine the equivalent elastic deflection, $X_E$.

$X_E = r_u/K_E = 369.5/652.5 = 0.566$ inch

Calculate the effective mass of the element.

$K_{LM} = 0.725$

$$w = \frac{11.1}{12} + \frac{3}{4} \times 18 \times \frac{490}{1{,}728} = (0.925 + 3.825)$$

$= 4.750$ lbs/in

Effective unit mass of element,

$$m_e = 0.725 \times \frac{4.75 \times 10^6}{32.2 \times 12}$$

$$= 0.891 \times 10^4 \text{ lbs-ms}^2/\text{in}^2$$

Calculate the natural period of vibration, $T_N$.

$$T_N = 2\pi\sqrt{(89.1 \times 10^2)/793} = 21.1 \text{ ms}$$

Determine the response parameters.
(Figure 6-7,
TM 5-1300)

Peak overpressure $B = 14.8 \times 36/2$

$= 266.5$ lbs/in

Peak resistance $r_u = 333.5$ lbs/in

Duration $T = 13.0$ ms

Natural period of vibration, $T_N = 21.1$ ms

$B/r_u = 266.5/333.5 = 0.799$

$T/T_N = 13/21.1 = 0.616$

From Figure 6-7, TM 5-1300,

$\mu = X_m/X_E = 1.1 < 3$ O.K.

$X_m = 1.1 \times 0.566 = 0.622$ in

$\tan \theta = X_m/(L/2) = 0.622/48 = .013$

$\theta = 0.75^\circ < 1^\circ$ O.K.

Check stresses at the connecting point.

$\sigma = My/I = 384.5 \times 10^3 \times (1.445 - 0.75)/24.881$

$= 10{,}740$ psi

$\tau = VQ/Ib$

$$= \frac{12.79 \times 10^3 \times 8 \times 3/4 \times (1.445 - 0.75/2)}{24.881 \times 1/2}$$

$= 5{,}321$ psi

Effective stress at the section $= \sqrt{\sigma^2 + \tau^2}$

$= 10^3 \times \sqrt{10.74^2 + 5.321^2}$

$= 10^3\sqrt{143.75}$

$= 11{,}980$ psi $< 36{,}000$ psi      O.K.

## 7.8 Design of Doubly-Symmetric Beams Subjected to Inclined Pressure-Time Loading

Problem 1: Design a purlin or girt as a flexural member which is subjected to a transverse pressure-time load acting in a plane other than a principal plane.

Procedure:

Step 1. Establish the design parameters.

a. Pressure-time load (TM 5-1300, Chapter 4)

b. Angle of inclination of the load with respect to the vertical axis of the section

c. Design criteria, $\mu_{max}$ and $\theta_{max}$ for a reusable or non-reusable structure (Section 2.3.3)

d. Member spacing, b

e. Type and properties of steel used:

Minimum yield strength for the section (Section 2.2.1)

Dynamic increase factor, c (Table 2.1)

Step 2. Preliminary sizing of the beam.

a. Determine the equivalent static load, w, using the following preliminary dynamic load factors:

$\text{DLF} = 1.0$ for reusable structure

$= 0.8$ for non-reusable structure

$w = \text{DLF} \times p \times b$

b. Using the appropriate resistance formula from Table 3.1 and the equivalent static load derived in Step 2a, determine the required $M_p$.

c. Determine the required section properties using Equation 3.2. Select a larger section since the member is subjected to unsymmetrical bending.

Note that for a load inclination of $10^{\circ}$, it is necessary to increase the required average section modulus, $(1/2)(S + Z)$, by 40 percent.

Step 3. Check local buckling of the member (Section 3.3.4).

Step 4. Calculate the inclination of the neutral axis (Equation 3.19).

Step 5. Calculate the elastic and plastic section moduli of the section (Equation 3.20).

Step 6. Compute the design plastic moment, $M_p$, (Equation 3.2).

Step 7. Calculate ultimate unit resistance, $r_u$, of the member.

Step 8. Calculate elastic deflection, $\delta$ (Figure 3.4).

Step 9. Determine the equivalent elastic unit stiffness, $K_E$, of the beam section using $\delta$ from Step 8.

Step 10. Compute the equivalent elastic deflection, $X_E$, of the member as given by $X_E = r_u/K_E$.

Step 11. Determine the load-mass factor, $K_{LM}$, and obtain the effective unit mass, $m_e$, of the element.

Step 12. Evaluate the natural period of vibration, $T_N$, (Equation 3.10).

Step 13. Determine the dynamic response of the beam. Evaluate $B/r_u$ and $T/T_N$, and use Figure 6-7 of TM 5-1300 or Figure 3.3 to obtain $X_m/X_E$ and $\theta$. Compare with criteria.

Step 14. Determine the ultimate dynamic shear capacity, $V_p$, (Equation 3.11) and maximum support shear, V, using Table 3.3 and check adequacy.

Example 8: Design an I-shaped beam for unsymmetrical bending due to inclined pressure-time loading.

Required: Design a reusable, simply-supported I-shaped beam subjected to a pressure-time loading acting at an angle of 10° with respect to the principal vertical plane of the beam.

Step 1. Given:

a. Pressure-time loading (Figure 7.8)

b. Design criteria: maximum ductility ratio = 1.5, maximum end rotation = 1°, whichever governs (Section 3.5)

c. Structural configuration (Figure 7.8)

d. Steel used: A36

Yield strength, $F_y$ = 36 ksi (Section 2.2.1)

Dynamic increase factor, c = 1.1 (Table 2.1)

Dynamic yield strength, $F_{dy}$ = 1.1 x 36 = 39.6 ksi (Equation 2.1)

Dynamic yielding stress in shear, $F_{dv} = 0.55F_{dy} = 0.55 \times 39.6 = 21.78$ ksi (Equation 2.2)

Modulus of elasticity, E = 29,000 ksi

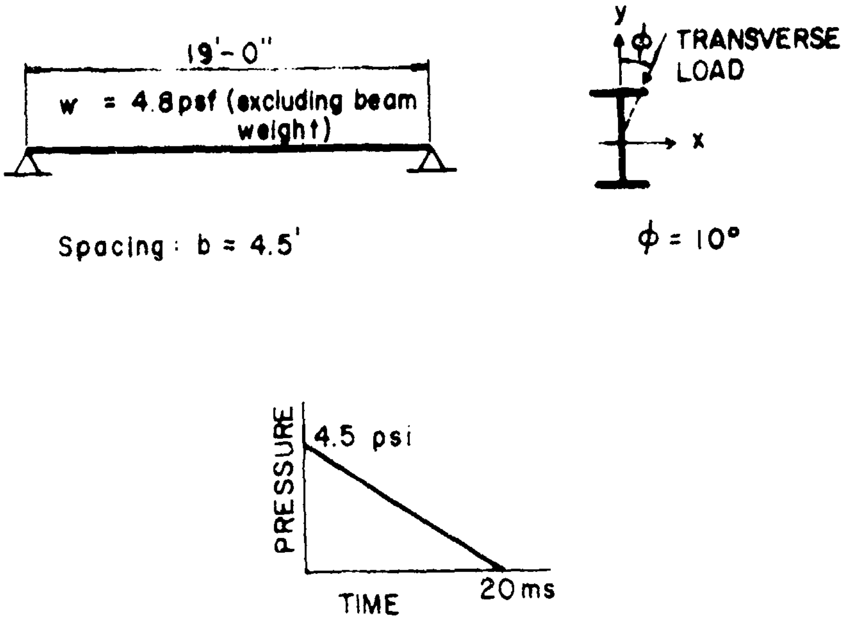
19'-0"
w = 4.8psf (excluding beam weight)
Spacing: b = 4.5'
y
ϕ
TRANSVERSE LOAD
x
ϕ = 10°
PRESSURE
4.5 psi
TIME
20ms

Figure 7.8 Beam configuration & loading, Example 8.

**Step 2. Preliminary sizing of the member.**

**a. Determine equivalent static load.**

**DLF = 1.0 (Section 3.3.2)**

**$w = 1 \times 4.5 \times 4.5 \times 144/1{,}000$**

**$= 2.92$ k/ft**

**b. Determine minimum required $M_p$**

**$M_p = (wL^2)/8 = (2.92 \times 19^2)/8$ (Table 3.1)**

**$= 132$ k-ft**

**c. Selection of a member.**

**For a load acting in the plane of the web,**

**$(S + Z) = 2M_p/F_{dy} = (2 \times 132 \times 12)/39.6$ (Equation 3.2)**

$(S + Z) = 80 \text{ in}^3$

$(S + Z)$ required $\approx 1.4 \times 80 = 112 \text{ in}^2$

Try W14 x 38, $S_x = 54.7 \text{ in}^3$, $Z_x = 61.6 \text{ in}^3$

$(S + Z) = 116.3 \text{ in}^3$, $I_x = 386 \text{ in}^4$

$I_y = 26.6 \text{ in}^4$

Step 3. Check against local buckling.

For W14 x 38, d = 14.12 inches,

$t_w = 0.313$ inch, $b_f = 6.776$ inches, and

$t_f = 0.513$ inch.

So, $d/t_w = 14.12/0.313$

$$= 45.2 < \frac{412}{\sqrt{36}}(1 - 1.4 \times \frac{P}{P_y})$$

$= 68.66$ O.K. (Equation 3.12)

$b_f/2t_f = 6.6776/(2 \times 0.513)$

$= 6.5 < 8.5$ O.K. (Section 3.3.4)

Step 4. Inclination of elastic and plastic neutral axes with respect to the x-axis.

$\tan \alpha = (I_x/I_y)\tan \phi$ (Equation 3.19)

$= (384/26.6) \tan 10^\circ$

$= 2.545$

$\alpha = 68.5^\circ$

Calculate the equivalent elastic section modulus.

$S = (S_x S_y)/(S_y \cos\phi + S_x \sin\phi)$

$S_x = 54.7 \text{ in}^3$, $S_y = 7.86 \text{ in}^3$, $\phi = 10^\circ$

$\sin 10^\circ = 0.174$, $\cos 10^\circ = 0.985$

$$S = (54.7)(7.86)/(7.86 \times 0.985 + 54.7 \times 0.174)$$

$$= 24.9 \text{ in}^3$$

Step 5. Calculate the plastic section modulus, Z.

$$Z = A_c m_1 + A_t m_2 \qquad \text{(Equation 3.20)}$$

$$A_c = A_t = A/2 = 11.2/2 = 5.6 \text{ in}^2$$

Let $\bar{y}$ be the distance of the c.g. of the area of cross-section in compression from origin as shown in Figure 7.9.

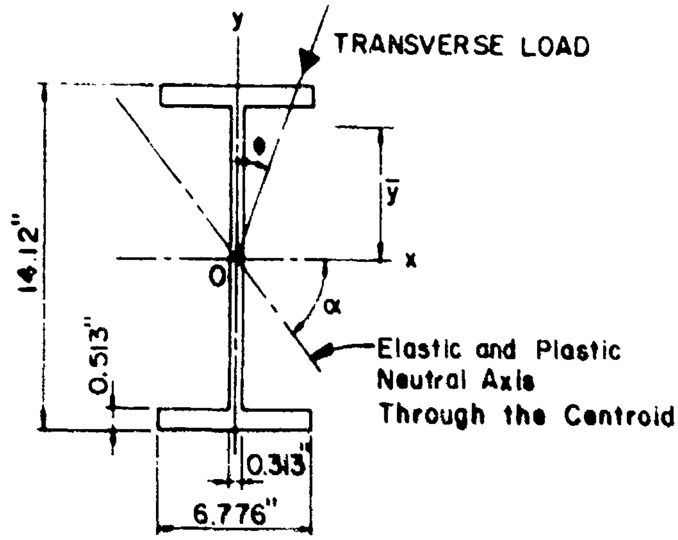
y
TRANSVERSE LOAD
θ
$\bar{y}$
x
O
α
14.12"
0.513"
0.313"
6.776"
Elastic and Plastic Neutral Axis Through the Centroid

Figure 7.9 Loading on beam section, Example 3.

$$y = \frac{1}{5.6}\left[6.776 \times 0.513 \times \left(\frac{14.12}{2} - \frac{0.513}{2}\right) + \frac{1}{2}(14.12 - 2 \times 0.513) \times 0.313 \times \frac{1}{2}\left(\frac{14.12}{2} - 0.513\right)\right]$$

$$= 5.42 \text{ inches}$$

$$m_1 = m_2 = y \sin \alpha = 5.42 \sin 68^\circ 30'$$

$$= 5.05 \text{ inches}$$

$$Z = 2A_c m_1 = 11.2 \times 5.05 = 56.5 \text{ in}^3$$

Step 6. Determine design plastic moment, $M_p$.

$$M_p = F_{dy}(S + Z)/2 = 39.6(24.9 + 56.5)/2 \qquad \text{(Equation 3.2)}$$

$$= 39.6 \times 40.7 = 1{,}612 \text{ in-kips}$$

Step 7. Calculate ultimate unit resistance, $r_u$.

$$r_u = 8M_p/L^2 \qquad \text{(Table 3.1)}$$

$$= (8)(1{,}612)(1{,}000)/(19 \times 12)^2$$

$$= 248 \text{ lbs/in}$$

Step 8. Compute elastic deflection, $\delta$.

$$\delta = \sqrt{(\delta_x^2 + \delta_y^2)} \qquad \text{(Section 3.5)}$$

$$\delta_y = \frac{5w\cos\phi L^4}{384EI_x}$$

$$\delta_x = \frac{5w\sin\phi L^4}{384EI_y}$$

$$w = \text{equivalent static load + dead load}$$

$$= 2.92 + \frac{(4.8 \times 4.5) + 38}{1{,}000} \text{ kips/ft}$$

$$= 2.94 \text{ kips/ft}$$

$$\delta = \left[\frac{(5w\sin\phi L^4)^2}{384EI_y} + \frac{(5w\cos\phi L^4)^2}{384EI_x}\right]^{1/2}$$

$$= \frac{5wL^4}{38{,}400E}\left[(0.652)^2 + (0.256)^2\right]^{1/2}$$

$$= 2.084 \text{ inches}$$

**Step 9.** Calculate the equivalent elastic unit stiffness, $K_E$.

$$K_E = w/\delta = \frac{2.94 \times 1{,}000 \times 1}{12 \times 2.085}$$

(Get w from Step 8)

$$= 117.8 \text{ lbs/in}^2$$

**Step 10.** Determine the equivalent elastic deflection, $X_E$.

$$X_E = r_u/K_E$$ (Equation 5-52, TM 5-1300)

$$= 248/117.8 = 2.11 \text{ inches}$$

**Step 11.** Calculate the effective mass of the element, $m_e$.

a. Load-mass factor, $K_{LM}$ (Table 6-1, TM 5-1300)

$K_{LM}$ (average elastic and plastic)

$$= (0.79 + 0.66)/2 = 0.725$$

b. Unit mass of element, m

$$m = w/g = \frac{[(4.5 \times 4.8) + 38] \times 10^6}{32.2 \times 12 \times 12}$$

$$= 1.286 \times 10^4 \text{ lbs-ms}^2/\text{in}^2$$

c. Effective unit mass of element, $m_e$

(Section 6-6, TM 5-1300)

$$m_e = K_{LM}m = 0.725 \times 1.286 \times 10^4$$

$$= 0.932 \times 10^4 \text{ lbs-ms}^2/\text{in}^2$$

**Step 12.** Calculate the natural period of vibration, $T_N$.

$$T_N = 2\pi\sqrt{(93.2 \times 10^2)/117.8}$$

(Equation 3.10)

$= 2\pi\sqrt{93.2/1.178}$

$= 2\pi\sqrt{79.1} = 55.8$ ms

Step 13. Determine the beam response.

Peak overpressure $B = 4.5 \times 4.5 \times 12$

$= 243$ lbs/in

Peak resistance $r_u = 248$ lbs/in

Duration $T = 20$ ms

Natural period of vibration, $T_N = 55.8$ ms

$B/r_u = 243/248 = 0.98$

$T/T_N = 20/55.8 = 0.358$

From Figure 6-7, TM 5-1300,

$X_m/X_E \approx 1$ O.K.

$X_m = 2.11$ inches

Find end rotation, θ.

$\tan\theta = X_m/(L/2) = 2.11/[(19 \times 12)/2]$

$= 0.0185$

$\theta = 1.06^\circ \approx 1.0^\circ$ O.K.

Step 14. Calculate the dynamic ultimate shear capacity, $V_p$, and check for adequacy.

$V_p = F_{dv}A_w = 21.78(14.12 - 2 \times 0.515)(0.313)$

$= 89.2$ kips (Equation 3.11)

$DLF = 1/(B/r_u) = 1/0.98 = 1.02$

$V = DLF \times B \times b \times L$

$= 1.02 \times 4.5 \times 4.5 \times 19 \times 144/(2 \times 1{,}000)$

$= 28.26$ kips $< 89.2$ kips $< V_p$ O.K.
(Table 3.5)

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# APPENDIX A

## MOMENT AND DEFLECTION COEFFICIENT CHARTS FOR UNIFORMLY-LOADED, TWO-WAY PLATE ELEMENTS

Charts are presented here for use in determining the elasto-plastic resistances, stiffnesses and deflections for uniformly-loaded two-way elements. The charts are to be used in conjunction with Figures 5-14 through 5-19 in Chapter 5 of TM 5-1300, "Structures to Resist the Effects of Accidental Explosions".

Figures A.1, A.2 and A.3 are for two-way elements with various support conditions on two adjacent edges with the remaining two edges free. Figure A.4 is for two-way elements with three edges fixed and one edge free, and updates and replaces Figure 5-14 of TM 5-1300. Figure A.5 is for two-way elements with two opposite edges fixed, one edge simply supported and one edge free, and is the complement of Figure 5-15. Figure A.6 is for elements with all edges fixed and replaces Figure 5-17 of TM 5-1300.

The use of these charts is detailed in Sections 5-13 and 5-14 of TM 5-1300. Although they were developed for particular values of Poisson's ratio, the charts can be employed with negligible error, for either steel plates ($\nu$ equal to 0.3) or concrete slabs ($\nu$ range from 0.15 to 0.25).

The symbols used in these charts are defined below:

$D$ = flexural rigidity per unit width (lb.in.$^2$/in.)

$H$ = height or width of plate (in.)

$L$ = length of plate (in.)

$M$ = moment per unit width (lb.in./in.)

$r$ = resistance per unit area (psi)

$X$ = transverse deflection (in.)

$\beta_i$ = moment coefficient for negative moment at point i

$\beta_V(max)$ = moment coefficient for maximum positive vertical moment

$\beta_H(max)$ = moment coefficient for maximum positive horizontal moment

$\gamma_i$ = deflection coefficient for point i

$\nu$ = Poisson's ratio

In the process of establishing the resistance functions for two-way elements with Figures A.1, A.2 and A.3, it may be found that the final resistance, as determined by using the procedures described in Chapter 5 of TM 5-1300, exceeds the total resistance, $R_u$, given by Table 5-6 in TM 5-1300. In this event, the elasto-plastic resistance diagram should be limited to reach its maximum resistance at $R_u$. This correction will not introduce any significant error into the bilinear resistance function developed from the elasto-plastic diagram.

Figures A.1 through A.5 are based upon finite element analyses and show good agreement with closed form solutions where such comparisons are possible. It appears, however, that the positive moment capacities as determined from Figures A.1 to A.3 are somewhat high as evidenced by the necessary slight adjustment described in the preceding paragraph. Overall, the charts provide necessary and sufficiently accurate data and should be employed in design until more refined results are developed.

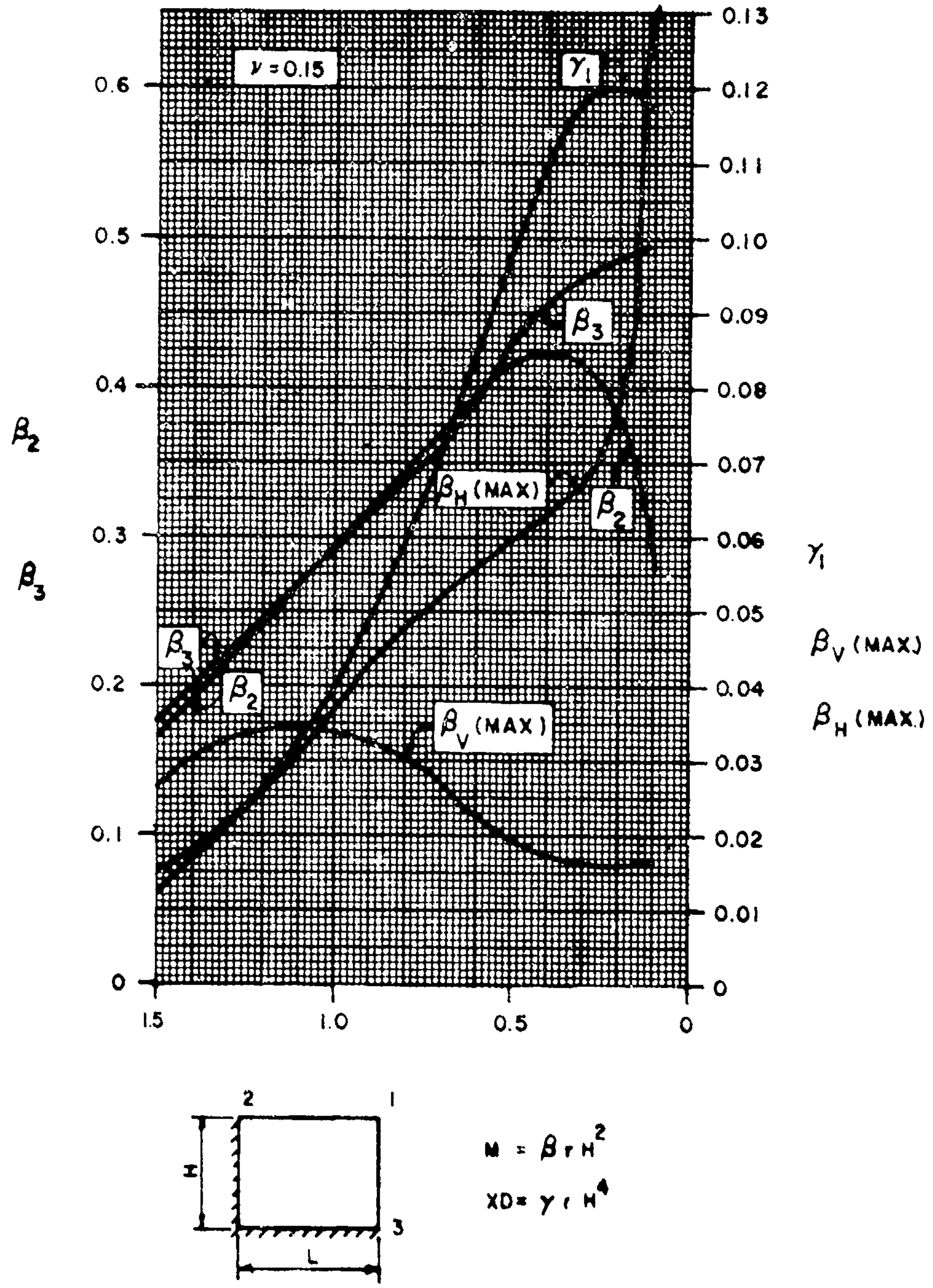


Figure A.1 Moment and deflection coefficients for uniformly-loaded, two-way element with two adjacent edges fixed and two edges free.

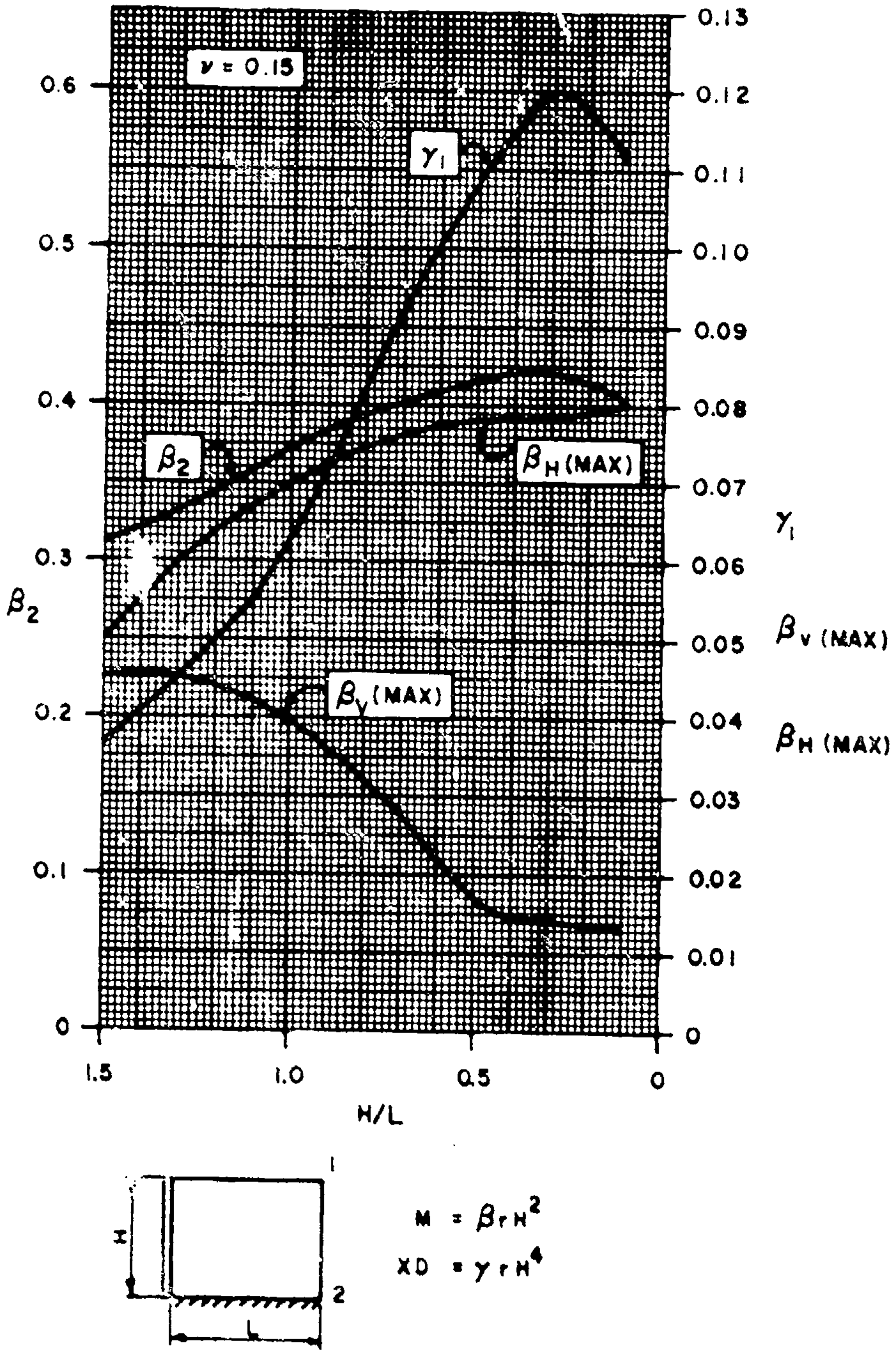


Figure A.2 Moment and deflection coefficients for uniformly-loaded, two-way element with one edge fixed, an adjacent edge simply-supported and two edges free.

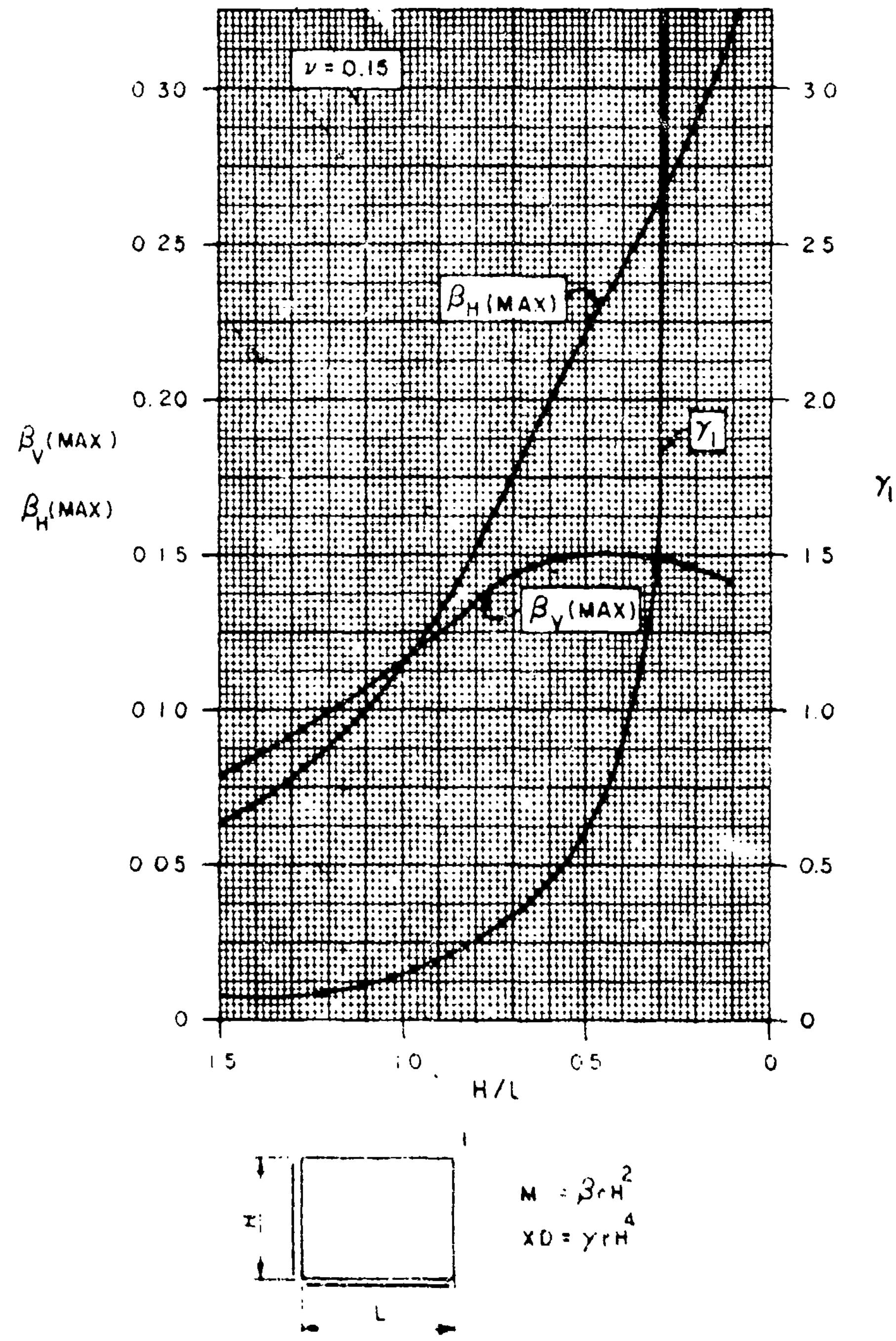
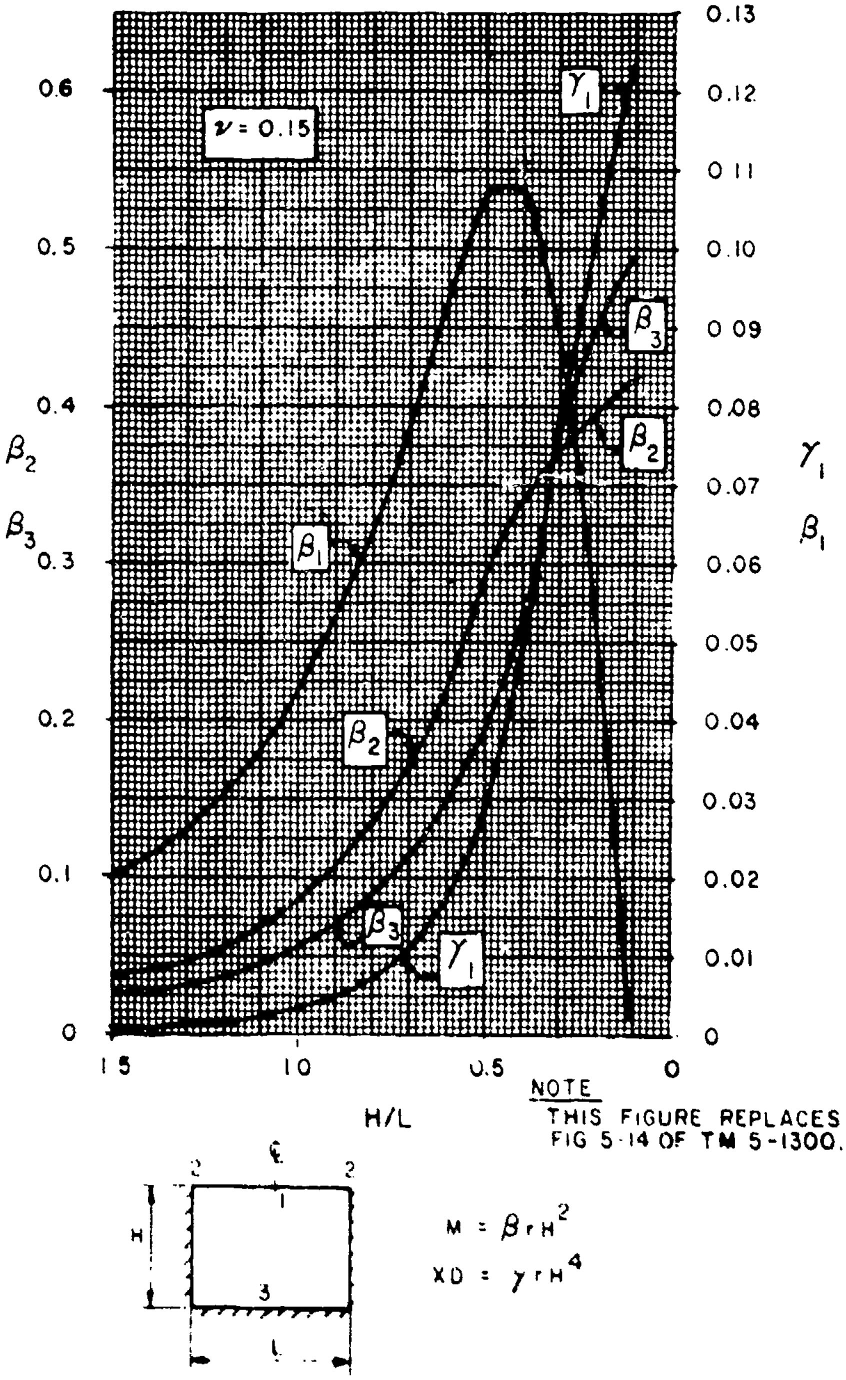


Figure A.3 Moment and deflection coefficients for uniformly-loaded, two-way element with two adjacent edges simply-supported and two edges free.



NOTE
THIS FIGURE REPLACES FIG 5-14 OF TM 5-1300.

Figure A.4 Moment and deflection coefficients for uniformly-loaded, two-way element with three edges fixed and one edge free.

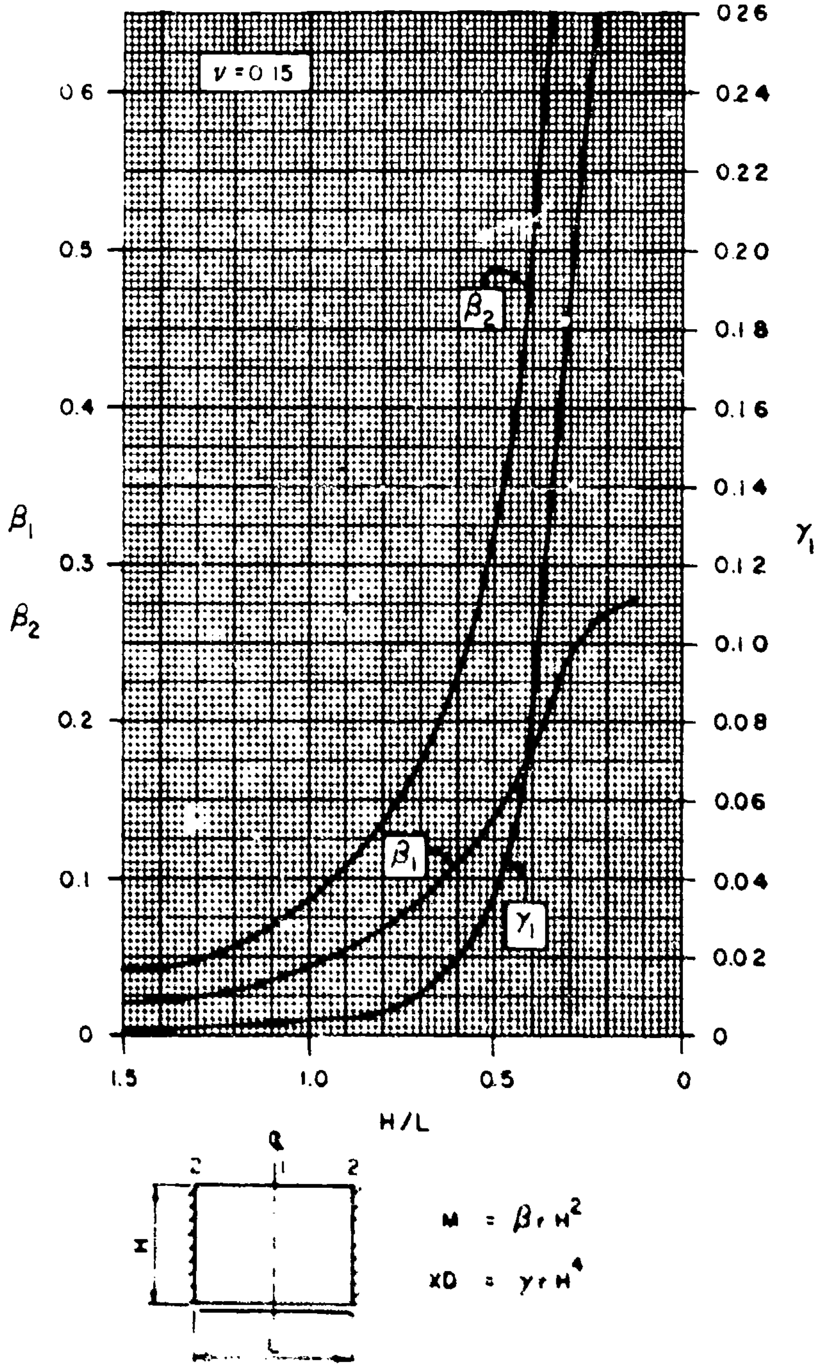


Figure A.5 Moment and deflection coefficients for uniformly-loaded, two-way element with two opposite edges fixed, one edge simply-supported and one edge free.

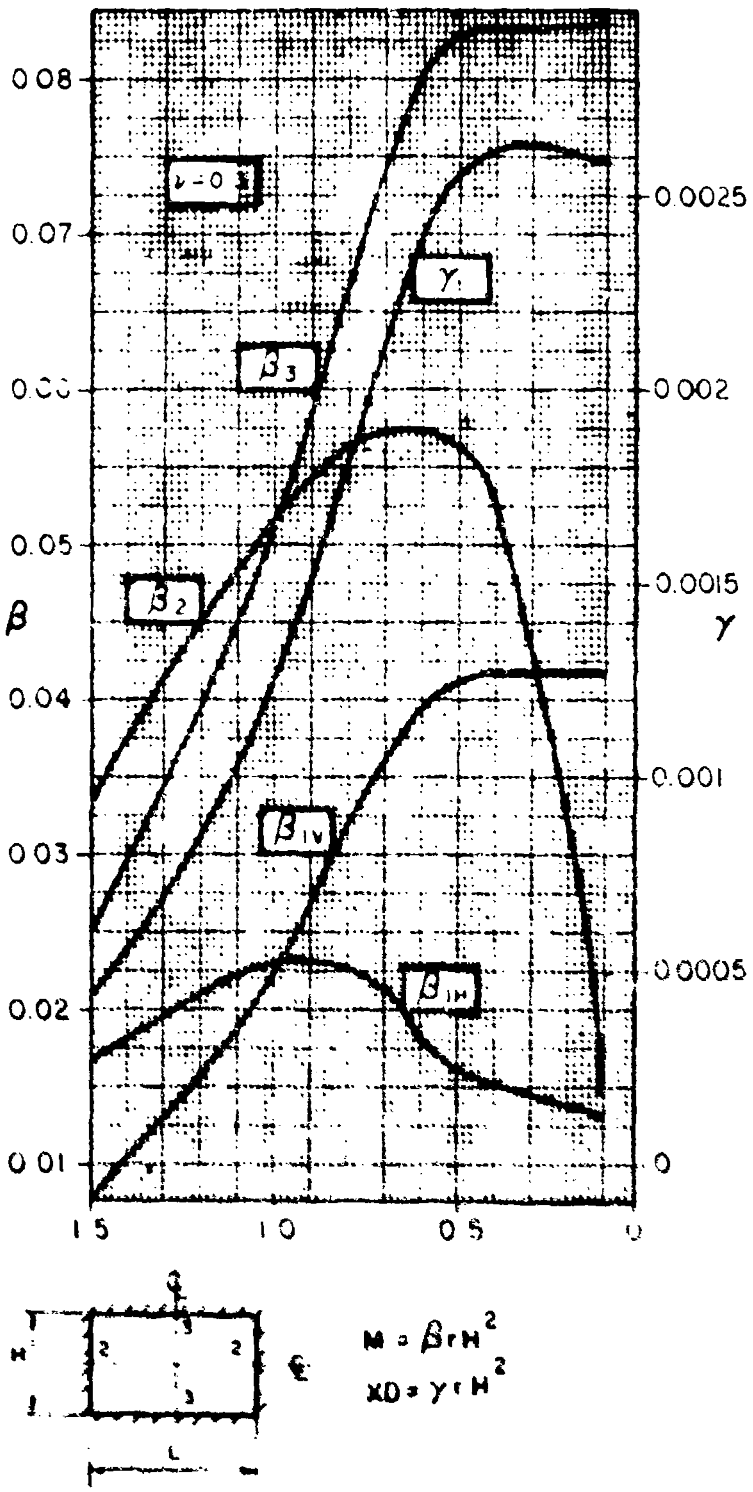


Figure A.6 Moment and deflection coefficients for uniformly-loaded, two-way element with all edges fixed.

## APPENDIX B

## LIST OF SYMBOLS

| | |
|---|---|
| $A$ | Area of cross-section (in$^2$) |
| $A_b$ | Area of bracing member (in$^2$) |
| $A_c$ | Area of cross-section in compression (in$^2$) |
| $A_t$ | Area of cross-section in tension (in$^2$) |
| $A_w$ | Web area (in$^2$) |
| $B$ | Peak pressure of equivalent triangular loading function, (psi) [when used with $r_u$], or peak total blast load (lb) [when used with $R_u$] |
| $b$ | Width of tributary loaded area (ft) |
| $b_f$ | Flange width (in.) |
| $b_h$ | Tributary width for horizontal loading (ft) |
| $b_v$ | Tributary width for vertical loading (ft) |
| $c$ | Dynamic increase factor |
| $c$ | Distance from neutral axis to extreme fiber of cross-section in flexure (in.) |
| $C, C_1$ | Coefficients indicating relative column to girder moment capacity (Section 5.2.1) |
| $C_b$ | Bending coefficient defined in Section 1.5.1.4.6a of the AISC Specification |
| $C_c$ | Column slenderness ratio indicating the transition from elastic to inelastic buckling |
| $C_{mx}, C_{my}$ | Coefficients applied to the bending terms in interaction formula (AISC Specification Section 1.6.1) |
| $C_2$ | Coefficient in approximate expression for sidesway stiffness factor (Table 5.3) |
| $D$ | Coefficient indicating relative girder to column stiffness (Table 5.3) |

## LIST OF SYMBOLS (cont.)

| | |
|---|---|
| DLF | Dynamic load factor |
| $d$ | Web depth (in.) |
| $E$ | Young's modulus of elasticity (psi) |
| $F$ | Maximum bending stress (psi) |
| $F_a$ | Axial stress permitted in the absence of bending moment (psi) |
| $F_b$ | Bending stress permitted in the absence of axial force (psi) |
| $F_{cr}$ | Web buckling stress (psi) |
| $F_d$ | Maximum dynamic design stress for connections (psi) |
| $F_{dv}$ | Dynamic yielding shear stress (psi) |
| $F_{dy}$ | Dynamic yield stress (psi) |
| $F'_{ex}, F'_{ey}$ | Euler buckling stresses divided by safety factor (psi) |
| $F_H$ | Horizontal component of force in bracing member (lb) |
| $F_S$ | Allowable static design for connections |
| $F_y$ | Static yield stress or yield strength |
| $g$ | acceleration due to gravity (32.2 ft/sec$^2$) |
| $H$ | Story height (ft) |
| $h$ | Web depth for cold formed, light gage steel panel sections (in.) |
| $I$ | Moment of inertia (in.$^4$) |
| $I_{ca}$ | Average column moment of inertia for single-story multi-bay frame (in$^4$) |
| $I_e$ | Effective moment of inertia for cold formed section at a service stress of 20 ksi (in$^4$) |
| $I_{eq}$ | Equivalent elastic moment of inertia for cold-formed steel panels (in$^4$) |

## LIST OF SYMBOLS (Cont.)

| | |
|---|---|
| $I_x$ | Moment of inertia about the x-axis ($in^4$) |
| $I_y$ | Moment of inertia about the y-axis ($in^4$) |
| $K$ | Effective length factor for a compression member |
| $K_b$ | Horizontal stiffness of diagonal bracing (lb/ft) |
| $K_E$ | Equivalent elastic stiffness (lb/ft) |
| $K_L$ | Load factor |
| $K_{LM}$ | Load-mass factor |
| $K_M$ | Mass factor |
| $L$ | Span length (ft) |
| $L$ | Frame bay width (ft) |
| $l/r$ | Slenderness ratio |
| $l_b$ | Actual unbraced length in the plane of bending (in.) |
| $l_r$ | Critical unbraced length (in.) |
| $M$ | Available moment capacity (Equation 3.18) |
| $M_e$ | Total effective mass ($lb\text{-}ms^2/in$) |
| $M_{mx}, M_{my}$ | Moments about the x-and y-axis that can be resisted by member in the absence of axial load |
| $M_p$ | Design plastic moment capacity |
| $M_{p1}, M_{p2}$ | Design plastic moment capacities (Figure 3.2) |
| $M_{px}, M_{py}$ | Plastic bending moment capacities about the x- and y-axes |
| $M_{pu}$ | Ultimate dynamic moment capacity |
| $M_{up}$ | Ultimate positive moment capacity for unit width of panel |
| $M_{un}$ | Ultimate negative moment capacity for unit width of panel |

## LIST OF SYMBOLS (Cont.)

| | |
|---|---|
| $M_y$ | Moment corresponding to first yield |
| $m$ | Number of braced bays in multi-bay frame |
| $m$ | Unit mass (psi-$ms^2$/in) |
| $m_e$ | Effective unit mass (psi-$ms^2$/in.) |
| $M_1$ | Distance from plastic neutral axis to the centroid of the area in compression, in a fully-plastic section (in.) |
| $m_2$ | Distance from plastic neutral axis to the centroid of the area in tension in a fully plastic section (in.) |
| $N$ | Bearing length at support for cold-formed steel panel (in.) |
| $n$ | Number of bays in multi-bay frame |
| $P$ | Applied compressive load (lb) |
| $P_{ex}, P_{ey}$ | Euler buckling loads about the x- and y-axes |
| $P_p$ | Ultimate capacity for dynamic axial load, $AF_{dy}$(lb) |
| $P_y$ | Ultimate capacity for static axial load $AF_y$(lb) |
| $P_h$ | Reflected blast pressure on front wall (psi) |
| $P_v$ | Blast overpressure on roof (psi) |
| $Q_u$ | Ultimate support capacity (lb) |
| $q_h$ | Peak horizontal load on frame (lb/ft) |
| $q_v$ | Peak vertical load on frame (lb/ft) |
| $R$ | Equivalent total horizontal static load on frame (lb) |
| $R_u$ | Ultimate total flexural resistance (lb) |
| $r_b$ | Radius of gyration of bracing member (in.) |
| $r_T$ | Radius of gyration, Equation 3.16 (in.) |

## LIST OF SYMBOLS (Cont.)

| | |
|---|---|
| $r_u$ | Ultimate flexural unit resistance (psi) |
| $r_x$, $r_y$ | Radii of gyration about the x- and y-axes (in) |
| $\bar{r}$ | Required resistance for elastic behavior in rebound (psi) |
| S | Elastic section modulus ($in^3$) |
| $S_x$ | Elastic section modulus about the x-axis ($in^3$) |
| $S_y$ | Elastic section modulus about the y-axis ($in^3$) |
| $S^+$ | Effective section modulus of cold-formed section for positive moments ($in^3$) |
| $S^-$ | Effective section modulus of cold-formed section for negative moments ($in^3$) |
| T | Load duration (sec) |
| $T_N$ | Natural period of vibration (sec) |
| t | Thickness of plate element (in) |
| $t_f$ | Flange thickness (in) |
| $t_m$ | Time to maximum response (sec) |
| $t_w$ | Web thickness (in) |
| V | Support shear (lb) |
| $V_p$ | Ultimate shear capacity (lb) |
| W | Total weight (lb) |
| $W_c$ | Total concentrated load (lb) |
| $W_E$ | External work (lb-in) |
| $W_I$ | Internal work (lb-in) |
| w | Flat width of plate element (in) |
| w | Load per unit area (psi) |

## LIST OF SYMBOLS (Cont.)

| | |
|---|---|
| $w$ | Load per unit length (lb/ft) |
| $X_1$ | Deflection at maximum resistance (Figure 3.8(c)) |
| $X_E$ | Equivalent elastic deflection (in.) |
| $X_m$ | Maximum deflection (in.) |
| $X_u$ | Ultimate deflection (in.) |
| $Z$ | Plastic section modulus ($in^3$) |
| $Z_x, Z_y$ | Plastic section moduli about the x-and y-axes ($in^3$) |
| $\alpha$ | Angle between the horizontal principal plane of the cross-section and the neutral axis (deg.) |
| $\alpha$ | Ratio of horizontal to vertical loading on a frame |
| $\beta$ | Base fixity factor (Table 5.3) |
| $\beta$ | Support condition coefficient (Section 3.7.2) |
| $\gamma$ | Angle between bracing member and a horizontal plane (deg.) |
| $\delta$ | Total transverse elastic deflection (in.) |
| $\delta$ | Lateral (sidesway) deflection (in.) |
| $\epsilon$ | Strain (in./in.) |
| $\dot{\epsilon}$ | Strain rate (in./in./sec) |
| $\theta$ | Member end rotation (Section 2.3.2) |
| $\theta$ | Plastic hinge rotation |
| $\theta_{max}$ | Maximum permitted member end rotation |
| $\mu$ | Ductility ratio |
| $\mu_{max}$ | Maximum permitted ductility ratio |
| $\phi$ | Angle between the plane of the load and the vertical principal plane of the cross-section (deg.) |

## LIST OF SYMBOLS (Cont.)

| | |
|---|---|
| $\phi_E$ | Beam curvature corresponding to development of design plastic moment capacity |
| $\phi_y$ | Beam curvature corresponding to development of $M_y$ |

# APPENDIX C

## TYPICAL DETAILS FOR BLAST-RESISTANT STRUCTURES

This appendix presents several examples of typical framing connections, structural details and blast doors used in industrial installations designed to resist accidental blast loadings.

Such buildings are often rectangular in plan, two or three bays wide and four or more bays long. Figure C.1 shows an example of a typical framing plan for a single-story building designed to resist a pressure-time blast loading impinging on the structure at an angle with respect to its main axes. The structural system consists of an orthogonal network of rigid frames. The girders of the frames running parallel to the building length serve also as purlins and are placed, for ease of erection, on top of the frames spanning across the structure's width.

Figures C.2 to C.5 present typical framing details related to the general layout of Figure C.1. As a rule, the columns are fabricated without splices, the plate covers and connection plates are shop welded to the columns, and all girder to column connections are field bolted. A channel is welded on top of the frame girders to cover the bolted connections and prevent (avoid) interference with the roof decking. All of the framing connections are designed to minimize stress concentrations and to avoid triaxial strains. They combine ductility with ease of fabrication.

Figure C.6 shows typical cross-sections of cold-formed, light gage steel panels commonly used in industrial installations. The closed sections, which are composed of a corrugated hat section and a flat sheet, are used to resist blast pressures in the low to intermediate pressure range, whereas the open hat section is recommended only for very low pressure situations as siding or roofing material. A typical vertical section illustrates the attachment of the steel paneling to the supporting members. Of particular interest is the detail at the corner between the exterior wall and the roof, which is designed to prevent peeling of the decking that may be caused by negative pressures at the roof edge.

Figure C.7 gives some typical arrangements of welded connections for attaching cold-formed steel panels to their supporting elements. Type A refers to an intermediate support whereas Type B refers to an end support. It is recommended that the diameter of puddle welds be 3/4 of an inch minimum and should not exceed 1-1/2 inches because of space limitations in the panel valleys. For deeper panels, it is often necessary to provide two rows of puddle welds at the intermediate supports in order to

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resist the uplift forces in rebound. It should be noted that welds close to the hooked edge of the panel are recommended to prevent lifting of adjacent panels.

Figure C.8 shows an arrangement of bolted connections for the attachment of cold-formed steel panels to the structural framing. The bolted connection consists of the following: a threaded stud resistance welded to the supporting member, a square steel block with a concentric hold used as a spacer and a washer and nut for fastening. Figure C.9 presents a cross-section of that connection with all the relevant details along with information pertaining to puddle welds.

Figures C.10 and C.11 show details of blast doors. Figure C.10 presents a single-leaf door installed in a steel structure. The design is typical of doors intended to resist relatively low pressure levels. It is interesting to note that the door is furnished with its tubing frame to insure proper fabrication and to provide adequate stiffness during erection. In the case of Figure C.11 the double-leaf door with its frame is installed in place and attached to the concrete structure. In both figures details of hinges, latches, anchors and panic hardware are illustrated. It should be noted that the pins at the panic latch ends are made of aluminum in order to eliminate the danger of sparking, a hazard in ammunition facilities, which might arise from steel-on-steel striking.

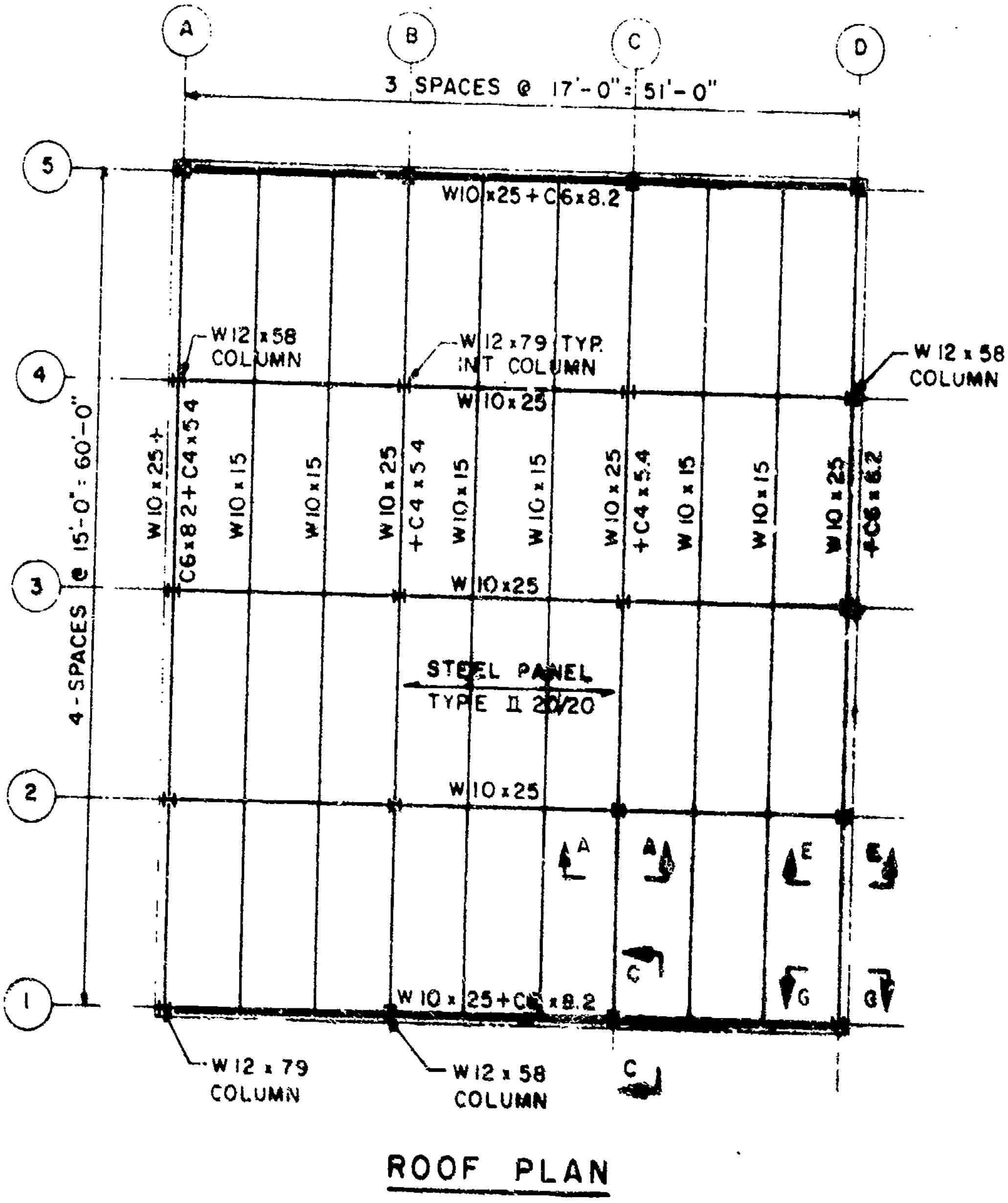
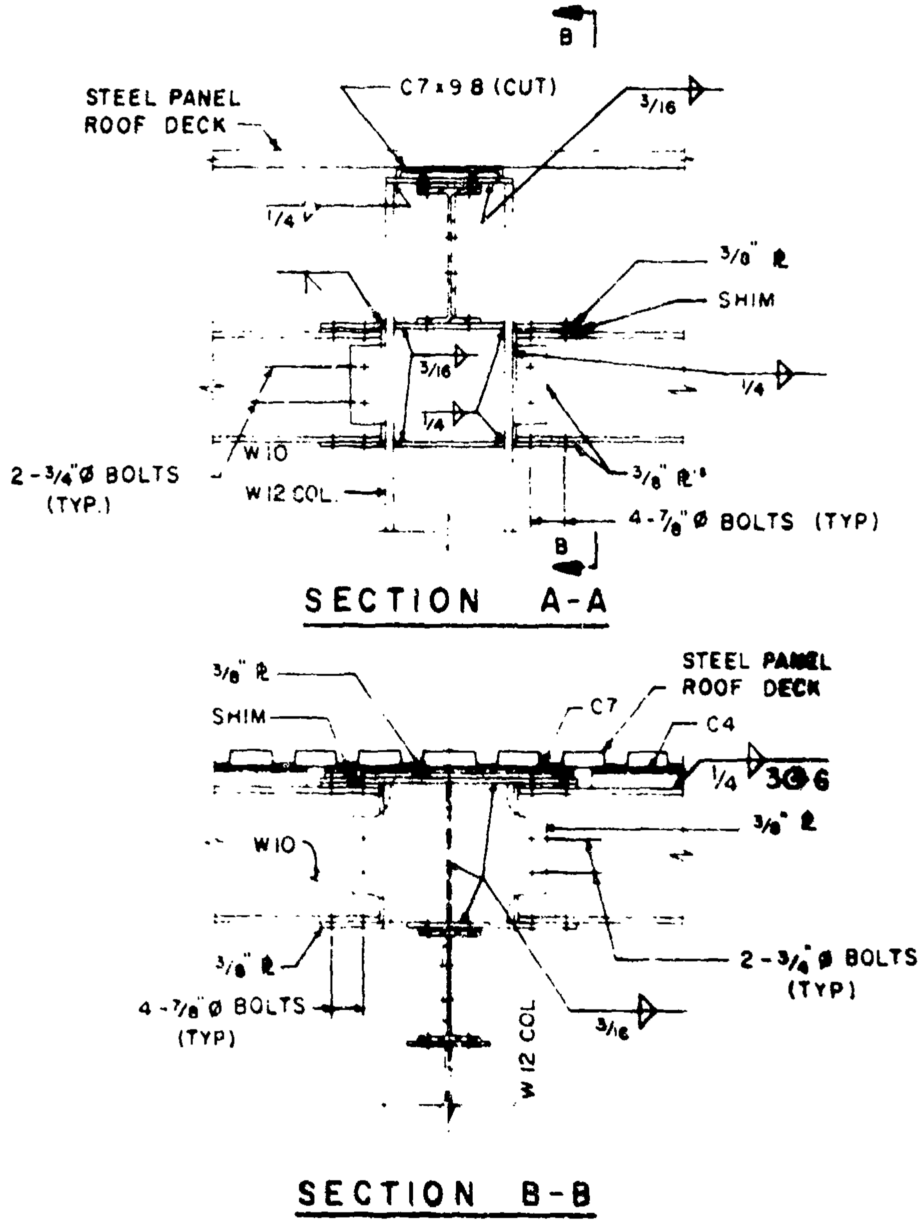

A
B
C
D
3 SPACES @ 17'-0" = 51'-0"
5
4
3
2
1
4-SPACES @ 15'-0" = 60'-0"
W10x25+C6x8.2
W12x58 COLUMN
W12x79 TYP. INT COLUMN
W12x58 COLUMN
W10x25
W10x25+C6x8.2+C4x5.4
W10x15
W10x15
W10x25 +C4x5.4
W10x15
W10x15
W10x25 +C4x5.4
W10x15
W10x15
W10x25 +C6x8.2
W10x25
STEEL PANEL
TYPE II 20/20
W10x25
A
A
E
E
C
G
G
W10x25+C6x8.2
W12x79 COLUMN
W12x58 COLUMN
C
ROOF PLAN

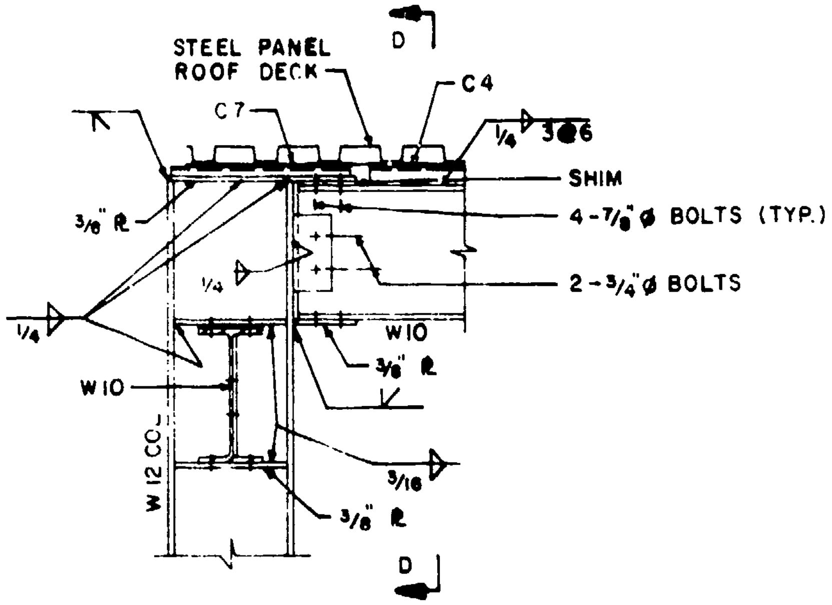
Figure C.1 Typical framing plan for a single-story blast-resistant steel structure.


SECTION A-A

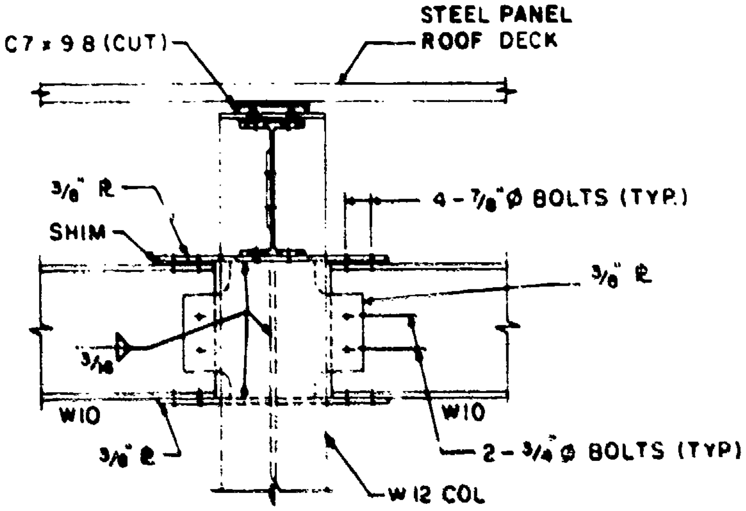
SECTION B-B

B
STEEL PANEL ROOF DECK
C7 x 9.8 (CUT)
3/16
1/4
3/8" ℞
SHIM
3/16
1/4
1/4
W10
2-3/4"Ø BOLTS (TYP.)
W12 COL.
3/8" ℞
4-7/8"Ø BOLTS (TYP)
B
3/8" ℞
SHIM
STEEL PANEL ROOF DECK
C7
C4
1/4 3@6
3/8" ℞
W10
3/8" ℞
2-3/4"Ø BOLTS (TYP)
4-7/8"Ø BOLTS (TYP)
W12 COL
3/16

Figure C.2 Typical framing detail at interior column ②-Ⓒ.

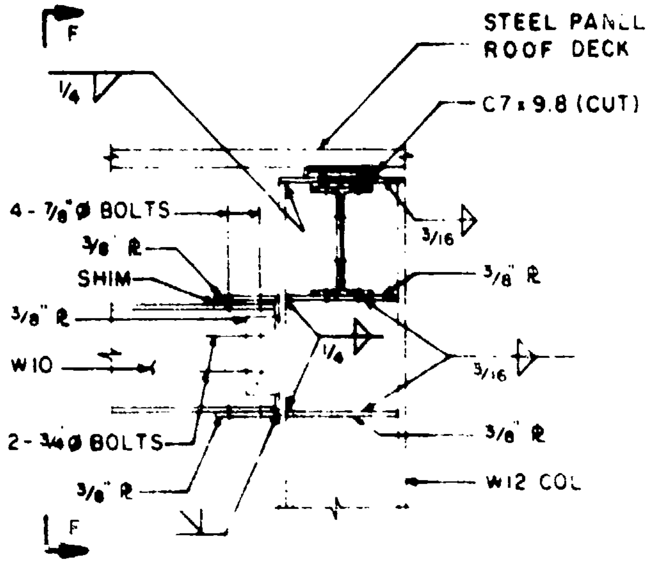

D
STEEL PANEL
ROOF DECK
C 7
C 4
1/4 3@6
SHIM
4 - 7/8" Ø BOLTS (TYP.)
3/8" PL
1/4
2 - 3/4" Ø BOLTS
1/4
W10
W 10
3/8" PL
W 12 COL
3/16
3/8" PL
D

SECTION C-C

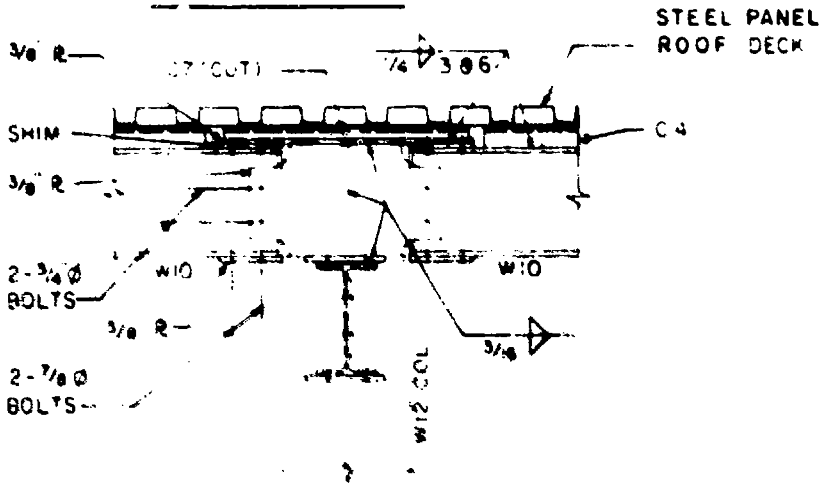

STEEL PANEL
ROOF DECK
C7 x 9.8 (CUT)
3/8" PL
SHIM
4 - 7/8" Ø BOLTS (TYP.)
3/8" PL
3/16
W10
W10
3/8" PL
2 - 3/4" Ø BOLTS (TYP)
W 12 COL

SECTION D-D

Figure C.3 Typical framing detail at end column ① - C1.

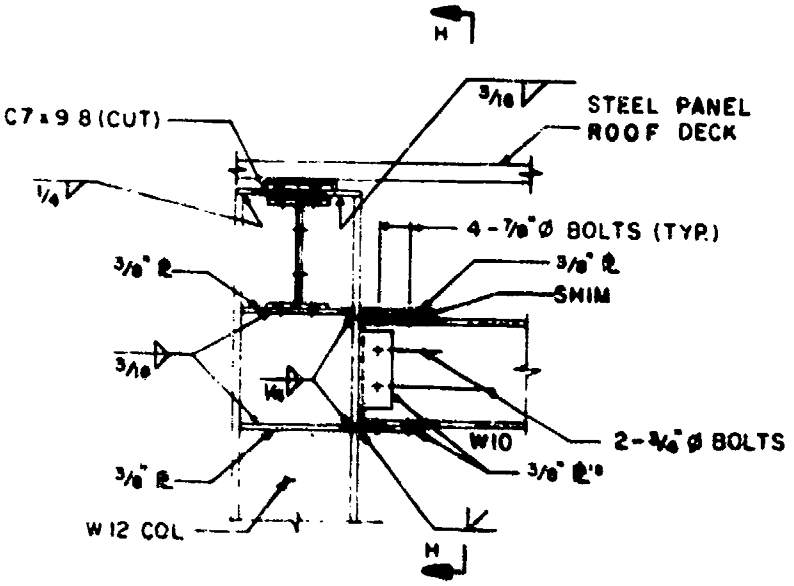

F
STEEL PANEL ROOF DECK
1/4
C7 x 9.8 (CUT)
4 - 7/8" Ø BOLTS
3/16
3/8" PL
SHIM
3/8" PL
3/8" PL
W10
1/4
3/16
2 - 3/4 Ø BOLTS
3/8" PL
3/8" PL
W12 COL
F

SECTION E-E

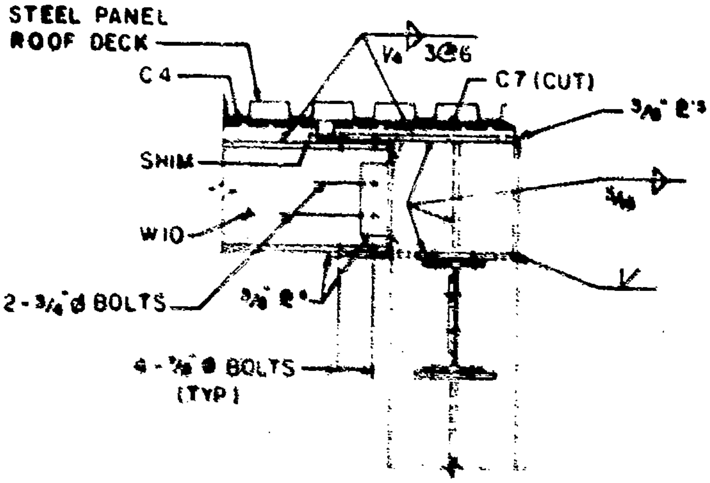

3/8" PL
C7 (CUT)
1/4 3 @ 6"
STEEL PANEL ROOF DECK
SHIM
C4
3/8" PL
2 - 3/4 Ø BOLTS
W10
W10
3/8 PL
3/16
2 - 7/8 Ø BOLTS
W12 COL

SECTION F-F

Figure C.4 Typical framing detail at side column 2 -10 .


H
3/16
STEEL PANEL ROOF DECK
C7 x 9.8 (CUT)
1/4
4 - 7/8"Ø BOLTS (TYP.)
3/8" PL
SHIM
3/16
1/4
2 - 3/4"Ø BOLTS
W10
3/8" PL's
3/8" PL
W12 COL
H

SECTION G-G


STEEL PANEL ROOF DECK
1/4 3@6
C4
C7 (CUT)
3/8" PL's
SHIM
3/16
W10
2 - 3/4"Ø BOLTS
3/8" PL's
4 - 7/8"Ø BOLTS (TYP)

SECTION H-H

Figure C.5 Typical framing detail at corner column (1 - D).

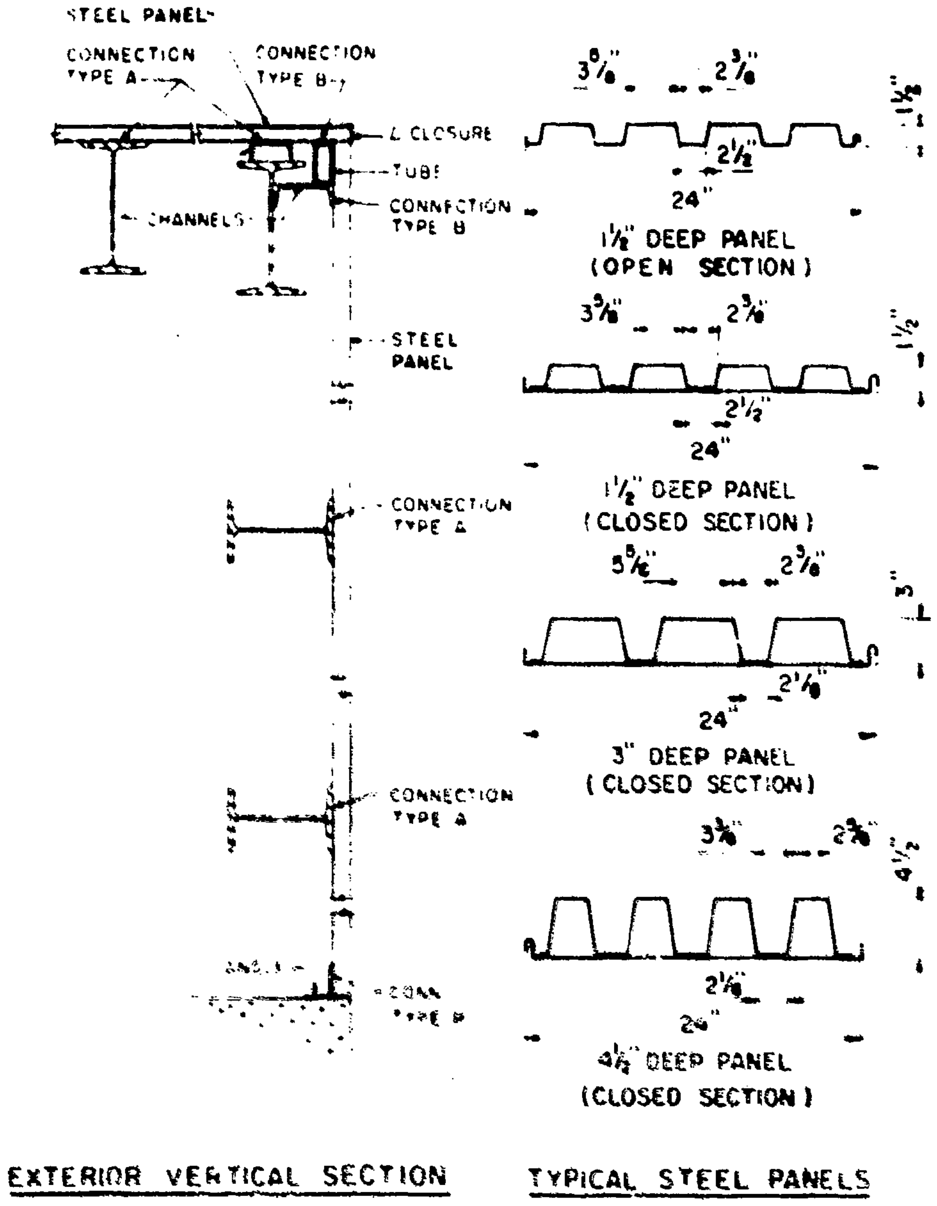
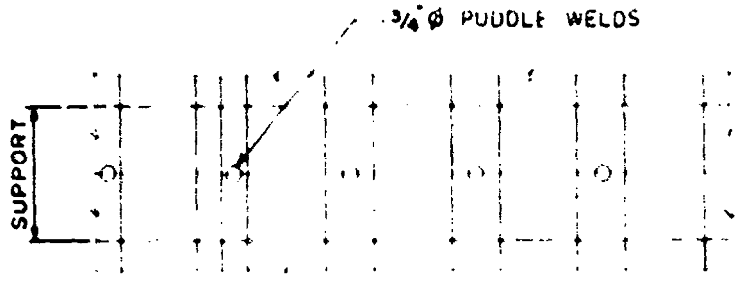
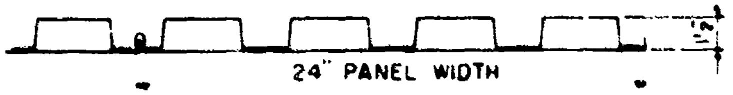

STEEL PANEL
CONNECTION TYPE A
CONNECTION TYPE B
℄ CLOSURE
TUBE
CHANNELS
CONNECTION TYPE B
STEEL PANEL
CONNECTION TYPE A
CONNECTION TYPE A
ANGLE
CONN. TYPE B
3⅝"
2⅞"
1½"
2½"
24"
1½" DEEP PANEL (OPEN SECTION)
3⅝"
2⅞"
1½"
2½"
24"
1½" DEEP PANEL (CLOSED SECTION)
5⅝"
2⅞"
3"
2⅛"
24"
3" DEEP PANEL (CLOSED SECTION)
3⅝"
2⅞"
4½"
2⅛"
24"
4½" DEEP PANEL (CLOSED SECTION)
EXTERIOR VERTICAL SECTION
TYPICAL STEEL PANELS

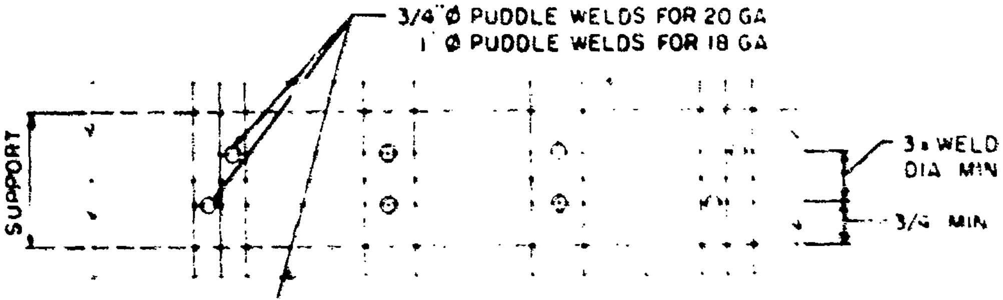
Figure C.6 Typical details for cold-formed, light gage steel paneling.


3/4" Ø PUDDLE WELDS
SUPPORT

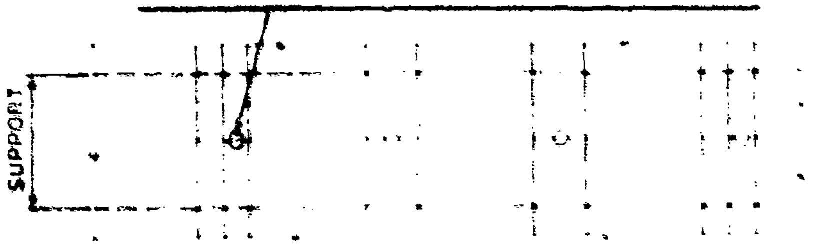
PLAN - WELDED CONNECTION TYPES A & B


24" PANEL WIDTH
1 1/2"

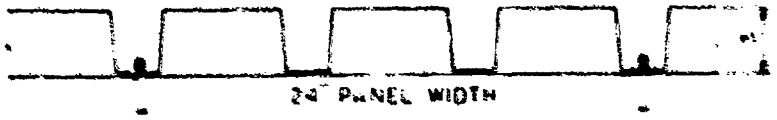
SECTION - 1 1/2" STEEL PANEL


3/4" Ø PUDDLE WELDS FOR 20 GA
1" Ø PUDDLE WELDS FOR 18 GA
SUPPORT
3 x WELD DIA MIN
3/4 MIN

PLAN - WELDED CONNECTION TYPE A



PLAN - WELDED CONNECTION TYPE B


24" PANEL WIDTH

SECTION - 3" STEEL PANEL

Figure C.7 Typical welded connections for attaching cold-formed steel panels to supporting members.

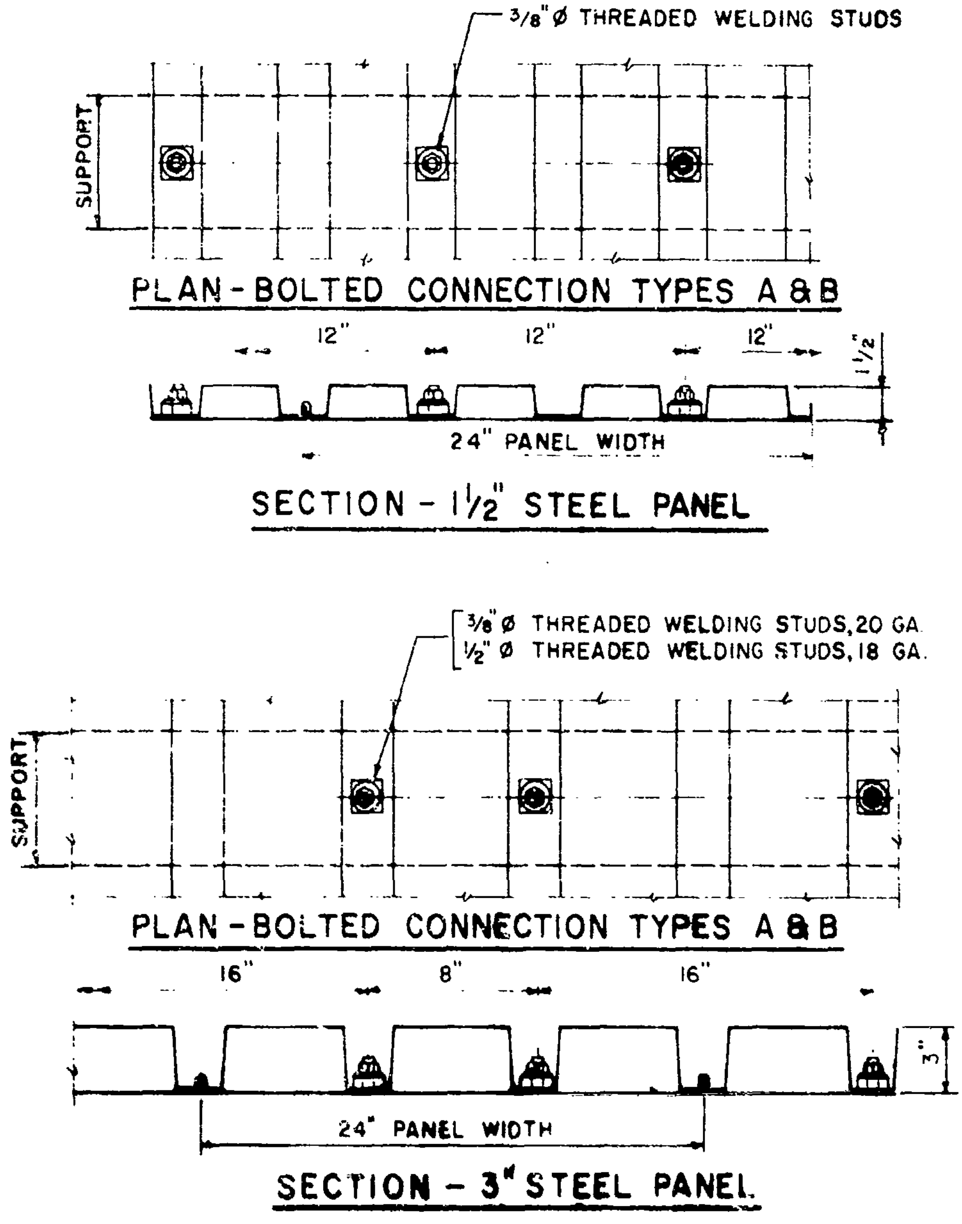
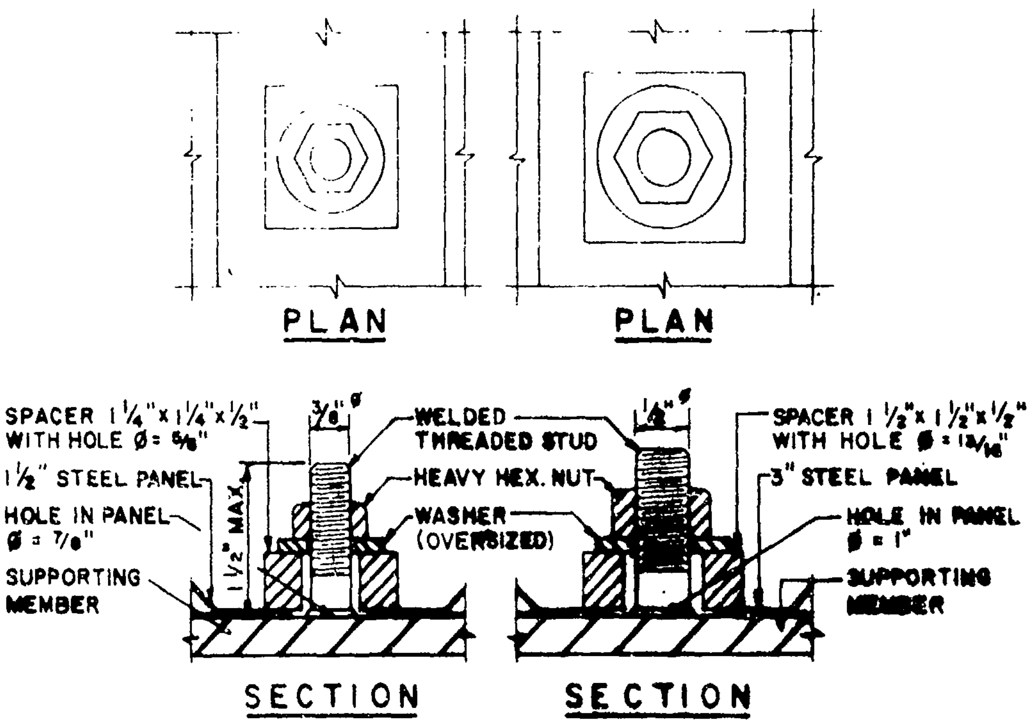
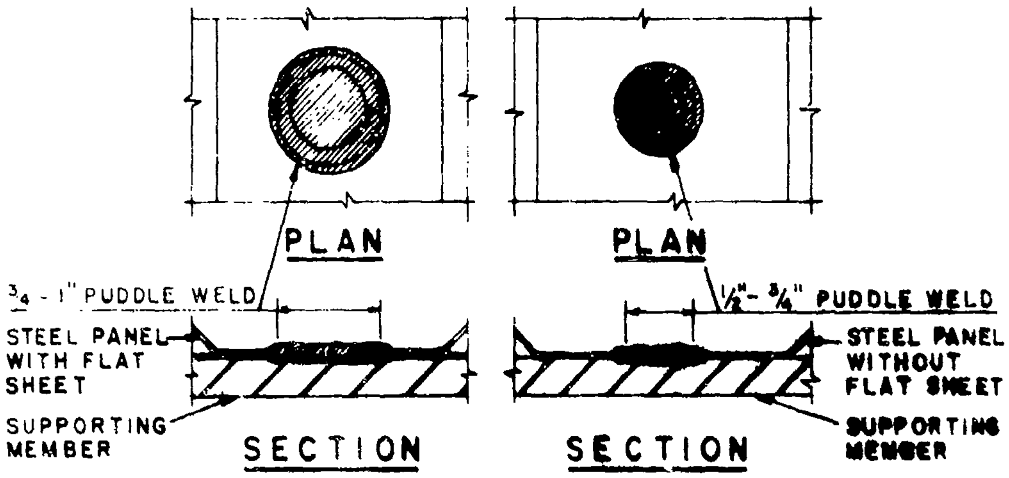

3/8"Ø THREADED WELDING STUDS
SUPPORT
PLAN - BOLTED CONNECTION TYPES A & B
12"
12"
12"
1 1/2"
24" PANEL WIDTH
SECTION - 1 1/2" STEEL PANEL
3/8"Ø THREADED WELDING STUDS, 20 GA.
1/2"Ø THREADED WELDING STUDS, 18 GA.
SUPPORT
PLAN - BOLTED CONNECTION TYPES A & B
16"
8"
16"
3"
24" PANEL WIDTH
SECTION - 3" STEEL PANEL

Figure C.8 Typical bolted connections for attaching cold-formed steel panels to supporting members.

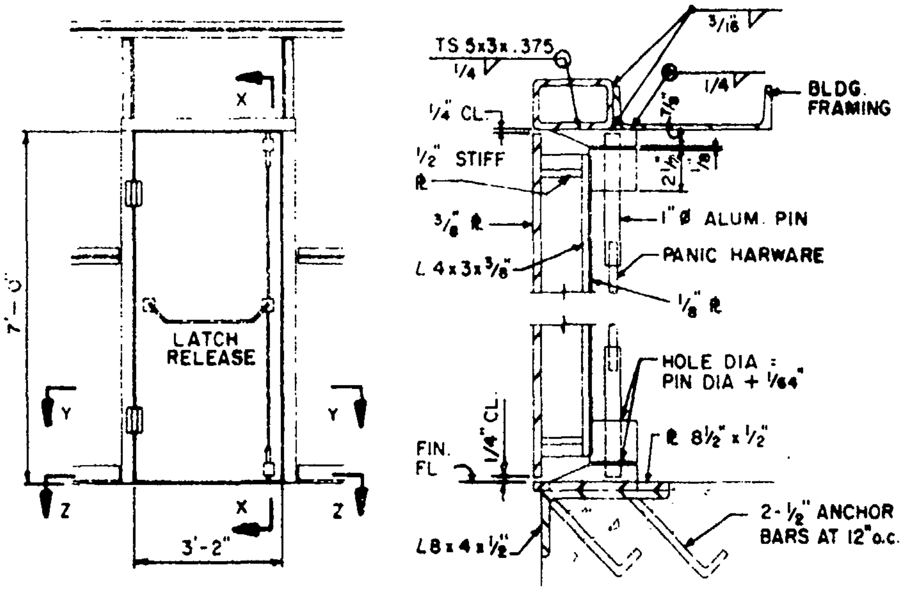

PLAN
PLAN
SPACER 1¼"x1¼"x½" WITH HOLE Ø = 5/8"
3/8"Ø
WELDED THREADED STUD
¼"Ø
SPACER 1½"x1½"x½" WITH HOLE Ø = 13/16"
1½" STEEL PANEL
HEAVY HEX. NUT
3" STEEL PANEL
HOLE IN PANEL Ø = 7/8"
1½" MAX.
WASHER (OVERSIZED)
HOLE IN PANEL Ø = 1"
SUPPORTING MEMBER
SUPPORTING MEMBER
SECTION
SECTION

TYPICAL BOLTED CONNECTION


PLAN
PLAN
¾ - 1" PUDDLE WELD
½"- ¾" PUDDLE WELD
STEEL PANEL WITH FLAT SHEET
STEEL PANEL WITHOUT FLAT SHEET
SUPPORTING MEMBER
SUPPORTING MEMBER
SECTION
SECTION

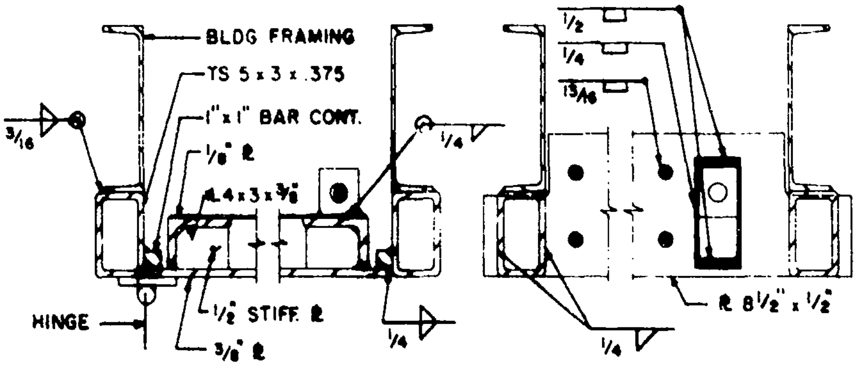
TYPICAL WELDED CONNECTION

Figure C.9 Details of typical fasteners for cold-formed steel panels.


X
7'-0"
LATCH RELEASE
Y
Y
Z
Z
X
3'-2"
TS 5x3x.375
1/4
3/16
1/4
BLDG. FRAMING
1/4" CL
1/2" STIFF PL
3/8" PL
L 4x3x3/8"
1" Ø ALUM. PIN
PANIC HARWARE
1/8" PL
2 1/2"
HOLE DIA = PIN DIA + 1/64"
PL 8 1/2" x 1/2"
FIN. FL
1/4" CL
L8x4x1/2"
2-1/2" ANCHOR BARS AT 12" o.c.

EXTERIOR ELEVATION

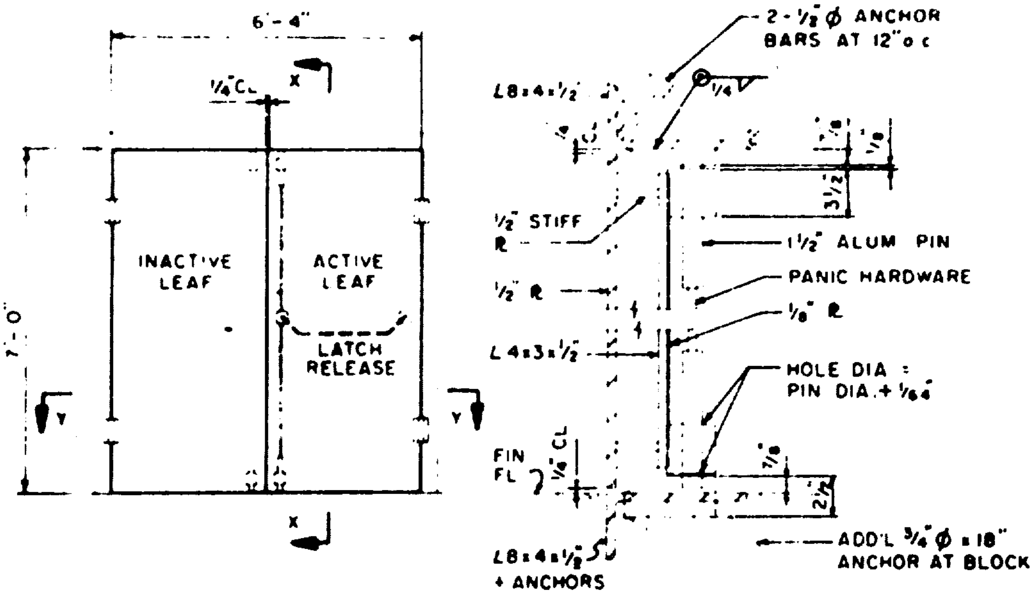
SECTION X-X


BLDG FRAMING
TS 5 x 3 x .375
1"x1" BAR CONT.
1/8" PL
L4x3x3/8"
3/16
1/4
HINGE
1/2" STIFF. PL
3/8" PL
1/4
1/2
1/4
13/16
PL 8 1/2" x 1/2"
1/4

SECTION Y-Y

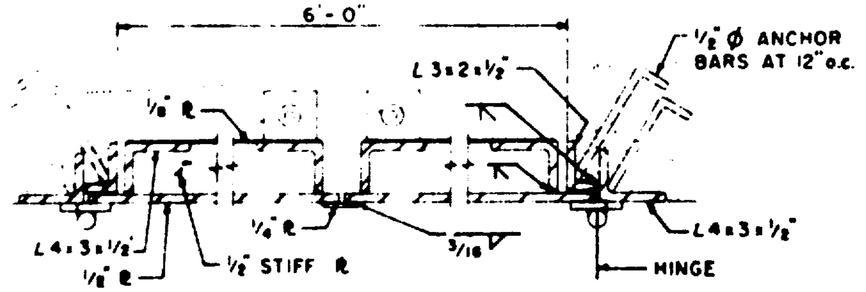
SECTION Z-Z

Figure C.10 Single-leaf blast door installed in a steel structure.


6'-4"
1/4" CL
X
7'-0"
INACTIVE LEAF
ACTIVE LEAF
LATCH RELEASE
Y
Y
X
2-1/2" ϕ ANCHOR BARS AT 12" o.c.
L8x4x1/2"
1/4
1/2" STIFF ℞
1/2" ℞
L4x3x1/2"
FIN FL
1/4" CL
L8x4x1/2" + ANCHORS
3 1/2"
1 1/2" ALUM PIN
PANIC HARDWARE
1/8" ℞
HOLE DIA = PIN DIA. + 1/64"
2 1/2"
ADD'L 3/4" ϕ x 18" ANCHOR AT BLOCK

EXTERIOR ELEVATION

SECTION X-X


6'-0"
1/2" ϕ ANCHOR BARS AT 12" o.c.
L3x2x1/2"
1/8" ℞
L4x3x1/2"
1/8" ℞
1/4" ℞
1/2" STIFF ℞
3/16
L4x3x1/2"
HINGE

SECTION Y-Y

Figure C.11 Double-leaf blast door installed in a concrete structure.

Best Available Copy